December 3, 2010

Redacted Version of the non-concurrence package – (a) Dissenting View on the AP1000 Shield Building Safety Evaluation Report With Respect to the Acceptance of Brittle Structural Module to be used for the Cylindrical Shield Building Wall; (b) Staff Response to the Dissenting View; and (c) Rebuttal to Staff Response on Dissenting View on the Safety Evaluation Report on the Design of the AP1000 Shield Building

Redactions are based upon staff acceptance of Westinghouse request for withholding of proprietary information, submitted in a November 22, 2010 letter to NRC (ML103340077).

U.S. NUCLEAR REGULATORY COMMISSION

NON-CONCURRENCE PROCESS

SECTION A - TO BE COMPLETED BY NON-CONCURRING INDIVIDUAL

TITLE OF DOCUMENT	ADAMS ACCESSION NO.
Safety Evaluation Report for AP1000 Shield Building	ML102630229
DOCUMENT SPONSOR Thomas Bergman	SPONSOR PHONE NO. 301-415-7192
NAME OF NON-CONCURRING INDIVIDUAL John S. Ma	PHONE NO. 301-415-2732

☐ DOCUMENT AUTHOR ☑ DOCUMENT CONTRIBUTOR ☑ DOCUMENT REVIEWER ☑ ON CONCURRENCE

TITLE	ORGANIZATION
Senior Structural Engineer	NRO/DE/SEB1

REASONS FOR NON-CONCURRENCE

The staff's proposed safety evaluation of the AP1000 shield building does not provide reasonable assurance of structural integrity for design basis events.

The AP1000 shield building safety functions include shielding plant structures from impact loads, such as tornado missiles, protecting plant equipment during seismic events, similar to containment and shield structures in other reactor designs. The shield building additionally supports the passive containment cooling water storage tank (PCCWST), and defining the cooling flow path for passive containment cooling. These additional functions are very important to the passive safety design, and are unique to the AP1000.

NRC regulations, such as GDC 1, require that structures, systems, and components important to safety be designed, fabricated, erected, and tested to quality standards commensurate with the importance of the safety functions to be performed.

NRC regulatory guidance endorses use of the American Concrete Institute (ACI) code for design of safety related structures at nuclear power plants. The ACI code is used as the basis for review of all other reinforced concrete structures, other than containment structures.

The applicant's design for AP1000 shield building does not fulfill key elements of the ACI code. Structural integrity cannot be assured for design basis events, because it has not been demonstrated that the building can absorb and dissipate energy imparted on the structure by an impact or seismic event. NRC staff acceptance of a design which does not fulfill the code would set a regulatory standard below that applied to other designs, in spite of the greater importance of the shield building to the AP1000 design.

Details of the basis for this position are described in the attached position papers:

Enclosure 1 (ADAMS Accession # ML103020232)
Enclosure 3 (ADAMS Accession # ML103081056)

☐ CONTINUED IN SECTION D

SIGNATURE	DATE
John S. Ma	11/04/2010

SUBMIT FORM TO DOCUMENT SPONSOR AND COPY TO YOUR IMMEDIATE SUPERVISOR AND DIFFERING VIEWS PROGRAM MANAGER

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NRC MD 10.158
(3-2009)

U.S. NUCLEAR REGULATORY COMMISSION

NON-CONCURRENCE PROCESS

TITLE OF DOCUMENT	ADAMS ACCESSION NO.
Safety Evaluation Report for AP1000 Shield Building	ML102630229

SECTION B - TO BE COMPLETED BY NON-CONCURRING INDIVIDUAL'S SUPERVISOR
(THIS SECTION SHOULD ONLY BE COMPLETED IF SUPERVISOR IS DIFFERENT THAN DOCUMENT SPONSOR.)

NAME
Brian Thomas

TITLE	PHONE NO.
Branch Chief	301-415-2803

ORGANIZATION
NRO/DE/SEB1

COMMENTS FOR THE DOCUMENT SPONSOR TO CONSIDER

☐ I HAVE NO COMMENTS

☑ I HAVE THE FOLLOWING COMMENTS

1) Refer to comments in Enclosure 2 (ADAMS Accession # ML103020239)

2) On the basis of comments received, staff is clarifying its evaluation of the Westinghouse Electric Company's pushover analysis under Section 4.10 Global Stability Analysis in the Safety Evaluation Report.

☐ CONTINUED IN SECTION D

SIGNATURE	DATE
	11/08/2010

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NON-CONCURRENCE PROCESS

TITLE OF DOCUMENT	ADAMS ACCESSION NO.
Safety Evaluation Report for AP1000 Shield Building	ML102630229

SECTION C - TO BE COMPLETED BY DOCUMENT SPONSOR

NAME Thomas Bergman	
TITLE Director	PHONE NO. 301-415-7192
ORGANIZATION NRO/DE	

ACTIONS TAKEN TO ADDRESS NON-CONCURRENCE (This section should be revised, as necessary, to reflect the final outcome of the non-concurrence process, including a complete discussion of how individual concerns were addressed.)

Actions to be taken in response to non-concurrence:

1. The concerns raised by the non-concurrence do not require revision to the document, "AP1000 Design Control Document, Revision 17 Safety Evaluation Report for Standard Review Plan Section 3.8.4 Other Seismic Category I Structures: Design of the Shield Building" (SER).

In summary, I agree with the staff's determination that the AP1000 shield building design meets regulatory requirements; specifically, in Title 10 of the Code of Federal Regulations (CFR) Part 50: Appendix A General Design Criteria (GDC) 1, 2 and 4, Appendix S, and 50.55a. Changes to the design that would satisfy the non-concurrence would result in a more robust structure. However, these changes are not necessary to meet regulatory requirements.

2. As described in Section B of this Non-concurrence Process Form, certain sections of the SER will be revised to clarify the bases for the staff's determination that there is reasonable assurance that the shield building meets relevant requirements. As these changes to the SER do not affect the design or analyses submitted by the applicant, nor address the issues raised in the non-concurrence, there is no need to seek re-concurrence by the non-concurring individual in the revised SER as his concerns remain unresolved.

Final status of the non-concurrence: The non-concurrence remains in place.

Bases for decision

The contributions of the non-concurring individual and other NRC staff improved the safety of the design that was originally submitted by the applicant and must be recognized. This application was a first-of-a-kind design. It relies on steel-concrete composite construction in a safety-critical application to an extent never before reviewed by the NRC. The design is further complicated by complex connections between the reinforced concrete walls and steel plate composite concrete walls, the basemat and the shield building, and between other structures and the shield building. It also includes a roof structure that incorporates a heavy passive containment cooling system (PCCS) water tank. The loads and utilization of both reinforced concrete and composite structures required careful analysis to establish the acceptability of this unique design. As a result of the NRC staff's efforts, the design has undergone substantial improvement: the addition of tie-bars and thicker steel plates throughout the composite structures, improved connections between the reinforced concrete and composite sections of the design, a strengthened tension ring and air-inlet region to handle the loads imposed by the roof structure and other demands, and a re-designed roof structure. Unfortunately, we were unable to achieve a consensus that the design has improved enough.

☑ CONTINUED IN SECTION D

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[signature]	11/4/2010	[signature]	11/04/2010

NON-CONCURRING INDIVIDUAL *(To be completed by document sponsor* when process is complete, i.e., after document is signed):

☐ CONCURS
☑ NON-CONCURS
☐ WITHDRAWS NON-CONCURRENCE (i.e., discontinues process)

☑ WANTS NCP FORM PUBLIC
☐ WANTS NCP FORM NON-PUBLIC

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SECTION D: CONTINUATION PAGE

CONTINUATION OF SECTION ☐ A ☐ B ☑ C

The discussion below summarizes the evaluation of the non-concurrence. The evaluation relied on both documentation review and meetings with staff, including the non-concurring individual. The documentation reviewed included that submitted in Sections A and B of this non-concurrence, the Office of Research's "Summary Evaluation Report" for the shield building, various emails and other correspondence submitted during the non-concurrence review, and the applicable portions of the applicant's design report for the shield building dated May 7, 2010, and supplemental information submitted September 3, 2010.

Following the concerns raised by the staff in the October 15, 2009, letter, the applicant began a redesign of the shield building to address the staff's concerns. The changes to be made to the shield building were to improve the roof structure, tension ring, air inlet region, connections between the reinforced concrete sections and composite sections, and modification of the composite design itself to ensure it behaved as a unit. There is a consensus that the changes to the roof, the tension ring, air inlet region, connections, and the composite module known as "Module 1" are acceptable. The non-concurrence concerns the composite module known as "Module 2."

Module 1 is used in areas of high loads, and is used in about 40% of the shield building walls. These areas are near the basemat where the connections between the reinforced concrete and composite construction occur, where there are connections to other structures, and in the air inlet region. Module 2 is used where loads are expected to be less, and is used in about 60 percent of the shield building walls. These areas are in the cylindrical portion of the shield building away from the basemat, below the air inlet region, and away from connections with other structures. The difference between the modules is in the spacing of the tie bars, Module 1 has close tie-bar spacing, about $1/6^{th}$ the thickness of the composite module (the air inlet region has other differences). Module 2 has greater spacing between tie bars, about ½ the thickness of the composite module. Except for the air inlet region, Module 1 and 2 have the same steel plate and overall thickness.

The applicant planned to demonstrate the acceptability of the modules through testing. Module 1 successfully passed its tests. The results for Module 2 were mixed. A Module 2 out-of-plane shear test failed in a brittle manner before the test could be completed. A Module 2 in-plane shear test could not be completed as a result of reaching the limitations of the test facility related to the amount of load being imposed on the test specimen. As a result, the ductility information on the Module 2 in-plane shear test was not obtained. The applicant later located comparable test data on steel-concrete composite structures to demonstrate acceptable performance of Module 2 for in-plane shear. The applicant performed cyclic testing on Module 2 and staff found that cyclic loading below yielding did not adversely affect out-of-plane stiffness or strength before failure. Therefore, Module 2 demonstrated adequate cyclic behavior even though it did not demonstrate a high level of ductility.

To supplement the test data, the applicant used confirmatory analysis to ensure adequacy of design assumptions. The applicant used commercial general-purpose computer codes to perform finite element method structural analyses. Models were developed at varying degrees of refinement, ranging from a very fine to a coarse mesh. The applicant performed both linear dynamic analyses and a nonlinear pushover analysis to predict the behavior of the shield building up to and beyond design basis seismic loading.

NON-CONCURRENCE PROCESS

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SECTION D: CONTINUATION PAGE

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At issue is whether the shield building can maintain its safety functions after design basis events (both earthquake and impact). There are three fundamental concerns in the non-concurrence that call into question the performance of the shield building following a design basis event. The first concern is lack of ductility of the Module 2 design that is used in about 60 percent of the shield building, mostly in the cylindrical section of the shield building. The second concern is that the applicant has not properly complied with applicable portions of the American Concrete Institute (ACI) 349 Code, "Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary," 2001. The third is that the analyses used to demonstrate the performance of the shield building are flawed both in terms of capability of the codes used and in their application to such a complex structure.

The lack of ductility in the Module 2 region for out-of-plane shear is a concern because it could lead to brittle failure in this region leading to loss of safety function of the shield building. Such a failure could occur at the point inelastic out-of-plane yielding begins in this region. However, on the basis of the applicant's analysis, the shield building response is elastic for the safe shutdown earthquake (SSE), the design basis requirement. Although not part of the Chapter 3.8 review, the structural response for the Review Level Earthquake (RLE) evaluated in Chapter 19 shows that although yield starts in a few locations of the wall at this load level, the strains are still small. For design basis impulse loads (e.g., tornado-generated missiles) the out-of-plane shear loads are well below those necessary to induce inelastic deformation. Although not part of the Chapter 3.8 review, the aircraft impact assessment performed by the applicant in accordance with 10 CFR 50.150 showed that there would be no perforation of the shield building due to impacts in the Module 2 region.

The applicant also conducted static analysis for loads well-beyond design basis loads (approximately 3 times the SSE) in the form of a "pushover" analysis. The purpose of the analysis was to confirm that inelastic deformation would initiate in the highly reinforced and ductile Module 1 region, and that out-of-plane shear loads in Module 2 would remain well below the out-of-plane shear capacity. The applicant referred to this approach as using the Module 1 region as a form of "plastic hinge" or "structural fuse" as a means to dissipate energy and limit loads in the non-ductile Module 2 region well below the strength of Module 2. The results of the analysis confirmed that the high stress areas of the shield building wall are the areas of the shield building where Module 1 is utilized. The applicant's tests showed that Module 1 exhibited ductile out-of-plane behavior under cyclic loading.

Collectively, the design basis and beyond design basis analyses conducted by the applicant demonstrate that out-of-plane shear is not a concern for design basis loads in the Module 2 region of the shield building. They also demonstrate that there is substantial margin in the design above design basis loads.

The second fundamental concern in the non-concurrence is that the applicant has not properly applied ACI-349. This concern is related to ductility, as the non-concurrence asserts that ACI-349 requires ductility in all structural components for seismically qualified structures. In addition, application of the Code would lead to different tie-bar spacing within Module 2, or would require test data to demonstrate the acceptability of an alternative spacing and would not allow reliance on analyses.

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ACI-349 is endorsed for use in Regulatory Guide (RG) 1.142, "Safety-Related Concrete Structures for Nuclear Power Plants (Other than Reactor Vessels and Containments)," and is widely used. ACI-349 was not developed for steel concrete composite structures like the shield building, although it would have been possible to modify the shield building design to conform to ACI-349 in the manner suggested by the non-concurrence. However, ACI-349 is not a regulatory requirement and, therefore, it is acceptable for the applicant to deviate from that Code.

In lieu of conformance to ACI-349, the applicant chose to take an alternative approach, as allowed, provided the applicant demonstrated that an acceptable level of safety would result. The approach taken by the applicant was a combination of use of ACI-349, test data, and analyses to demonstrate that the shield building met regulatory requirements. The applicant applied provisions of the Code: for example, use of capacity formulas in the design of the composite modules. As described above, there is reasonable assurance that the shield building meets regulatory requirements on the bases of the results of the alternative approach.

The third fundamental concern in the non-concurrence is the use of the analytical modeling codes used by the applicant. The applicant used commercial general purpose computer codes to perform the analyses. The non-concurrence raises two concerns: the use of general purpose computer codes for this application, and that the results of the analyses using the general purpose codes would be, at best, approximations.

The concern over the use of general purpose codes is that the shield building is complex in terms of the connections between reinforced concrete structures and the composite structures, and that these connections make it difficult to predict loads within the structure and introduce discontinuities into the analysis that make it difficult to model. The non-concurrence suggests that custom computer codes should have been developed to ensure the behavior of the modules and the connections were properly modeled and analyzed. Absent specific issues with the application of the general purpose codes used by the applicant (for example, improper modeling of a module or connection) that would result in a material erroneous result, we have no basis for rejecting the applicant's use of these general purpose codes.

The second issue is that the results of the general purpose codes are, at best, approximations. The applicant was aware of the limitations of these codes in their application to the shield building, and took a number of measures to ensure the results were appropriate. For example, the applicant used conservative limiting values for tie-bar tensile strain so that the analyses would terminate when such strains were reached. The applicant also provided benchmarking data to demonstrate that the computer codes could reasonably predict the behavior of tests. As noted above, the analyses showed that there was substantial margin above design basis loads in the design. The margin is so large that it would allow for substantial uncertainty and still result in the conclusion that the shield building meets regulatory requirements. Absent specific information that the results are materially inaccurate, we have no basis for rejecting the applicant's approach.

December 3, 2010

Enclosure 1

Redacted Version of Dissenting View on the AP1000 Shield Building Safety Evaluation Report With Respect to the Acceptance of Brittle Structural Module to be used for the Cylindrical Shield Building Wall

1. Introduction

A structure is safe only if it is designed and constructed correctly. A design is correct only if a proven design method is used and used correctly. A design method is proven to be correct only if it was derived from, or substantiated by, tests.

New structural modules (module #1 and module #2) have been proposed for use of the AP1000 shield building. The structural module consists of two steel faceplates with tie-wires connect the faceplates and concrete filled between them (SC wall module). A structural module with such a construction possesses high flexural strength (energy) perpendicular to the faceplates, the surface of the cylindrical shield building wall, but its shear strength and energy in the same direction (out-of-plane), which is needed to resist the out-of-plane shear force and energy generated by impact from tornado missiles or aircraft, or induced by ground motions from earthquakes, depends on the size and spacing of tie-wires.

Since out-of-plane shear failure is a prevailing failure mode for the shield building, Westinghouse (WEC) proposed test methods and acceptance criteria for both modules. The test methods and acceptance criteria were agreed and accepted by the NRC. [redacted]

[redacted] the NRC staff held a [redacted]e conference with its two consultants in seismic shear design. The two consultants and the staff agreed that the [redacted] of module #2 should not be accepted and a redesign for module #2 is warranted, because the purpose of the test was to substantiate whether the design (tie-wire spacing) of module #2 was sufficiently close so that the structural module possessed ductility meeting the acceptance criteria. SEB1 sent this feedback information through the project manager to WEC (see Attachment 1).

Attachment 1 also contained feedback information regarding problems of testing, construction, and welding. This is because the NRC also engaged a consultant, who is an expert in testing, a consultant, who is an expert in construction, and a consultant, who is an expert in steel welding, because the WEC shield building with the proposed use of new SC modules not only creates design problems but also problems in areas of testing, construction, and steel welding. These problems affect the integrity of the shield building, and need to be considered in the design. For an example, the staff was especially concerned about the constructability issue raised by its consultant, as stated in Attachment 1 "The proposed connection and air inlet and tension ring have constructability problems, such as steel rod alignment, aggregate size, air entrapment, bleed water accumulation, and design implications, such as elongation in rods, shear friction transfer, and compression force transfer. The staff has no confidence that a potential success of carefully mockup tests would be replicated during construction." Difficult construction usually causes an inferior quality of the structure, and thus requires a higher design margin than usual to compensate for the anticipated construction problem.

However, this write-up only addresses the design problem with respect to the adequacy of module #2, [redacted]

In the Rev.2 report submitted by WEC, Module #2 remained to be part of the SC wall without modification. The two NRC consultants disagreed with the use of module #2 [redacted] They reiterated the importance of using ductile structural modules for those structures subjected to earthquakes and warned that the ACI Code prohibited the brittle failure modes, such as shear failure. They recommended design changes to module #2 so that [redacted] would be eliminated for the shield building. The letter from one of the consultants is attached (see Attachment 2).

Westinghouse submitted a justification for continuing the use of module #2, dated September 3, 2010. Westinghouse claimed that the area of the SC wall module #2 (about 60% of the total SC wall) does not need ductility, and complies with ACI Code requirements. The author finds that the justification does not have technical merit, as explained below. However, the SEB1 SER accepts the justification and finds the module #2 acceptable. The author will explain and demonstrate that (1) the SC wall module #2 neither possesses the ductility required either based on the ACI Code's prescriptive requirements for RC structures, nor meets the agreed upon ductility criteria proposed for the SC wall modules by WEC, and (2) the SEB1 SER justifications for accepting module #2 are invalid, and its acceptance standard of assurance for safety is less than that of building code requirements in the United States that regulate the design for all reinforced concrete structures, including nuclear power plant structures, and does not comply with the intent of 10 CFR 50.55a (a)(1) and 10 CFR Part 50, Appendix A, GDC1.

Section 2 of this write-up describes the SC wall and the major problems associated with the design of the wall. Section 3 defines the meaning of four basic structural characteristics of a structural module or a structure: strength, stiffness, ductility, and energy absorption/dissipation capability, and provides a comparison of the four characteristics for the brittle SC wall module #2 and a RC wall module with nearly identical shear reinforcement (tie-bars), under out-of-plane shear tests. Section 4 describes WEC design basis. Section 5 describes NRC acceptance criteria. Section 6 describes the applicability of the ACI Code to the SC wall, and the intent and requirements of the Code. Section 7 describes the design change for module #2 recommended by the experts. Section 8 describes the author's position. Section 9 describes the WEC justification for the continuing use of the brittle SC wall module, and the author's evaluation. Section 10 states the reasons of non-concurrence by the author and describes possible solutions to address his concerns

2. Background

2.1 Description of the SC wall

A new type of structural wall modules was proposed by Westinghouse (WEC) for the AP1000 cylindrical shield building wall. The structural wall (SC) module is made from [redacted] the faceplates together and concrete fills between the faceplates. The auxiliary building roof is connected to the SC cylindrical portion of the shield building. The floor slabs and interior structural walls of the auxiliary building are also structurally connected to the RC cylindrical portion of the shield building. The SC wall is attached to the top and sides of the RC wall with stepped or "zigzagged" boundary conditions both in the vertical (meridional) and horizontal (hoop) directions (SB Report, Revision 2, Figure 3.2-2). The SC wall module steel faceplates are not anchored to the RC walls. The SC wall and the RC wall are connected together through [redacted] connectors (SB Report, Revision 2, Figures 4.1-2, 4.1-3, 4.1-4, and 4.1-5), and the SC wall is also connected to the reinforced concrete basemat through [redacted]

The West part of the [redacted] SC wall (West wall) extends, for the most part, from the top of the basemat (EL 100') to the bottom of the ring girder (EL 266' 3") while the East wall, that part of the [redacted] SC wall between azimuths 341.94 degrees and 174.60 degrees (a span of 192.66 degrees), intersects the auxiliary building and is structurally attached to the walls and slabs of that building. As a consequence, the configuration of the SC wall has significant irregularity.

The free standing vertical span of the West wall, height from the top of the basemat to the bottom of the tension ring, is 166' 3". The East part of the SC wall connects to the RC wall of the shield building (the part of the [redacted] thick wall protected by the auxiliary building structure) below the roof of the auxiliary building at elevation 146' 10". The RC floors and walls of the auxiliary building attach to the RC wall of the shield building and constrain lateral displacements of this wall. The height of the East wall above its SC/RC connection located below the roof of the auxiliary building is 119' 5". The free standing vertical span of the East wall, defined here as the height above the roof of the auxiliary building, is between 112' 3" and 103' 3" for most of the East wall, and 85' 6" for a short length of the wall near the 180 degrees azimuth. As a consequence of the above, the lateral stiffness of the SC East wall is greater than that of the SC West wall. Therefore, the stiffness of the SC wall is asymmetric in the hoop direction.

The part of the wall from the SC/RC connections to EL 248' 6 ½" consists of [redacted] thick SC modules. The part of the wall above the [redacted] thick SC wall up to the bottom of the tension ring at EL 266' 3" is the air inlet structure. The wall thickness for the air inlet region is [redacted] between EL 248' 6 ½" and EL 251' 6 ½", increases linearly from [redacted] between EL 251' 6 ½" and EL 254' 6 ½", and remains constant at [redacted]6 inches up to the bottom of the tension ring. Therefore, there is a stiffness change in the vertical direction of the shield building.

[redacted]

2.2 Major Problems of the SC wall

Analysis:

- Irregular configurations of the SC wall generate uncertain analytical results, especially with respect to torsion: As described above, the configuration of the shield building is highly irregular. Irregular configuration is considered to be the most undesirable design considerations, because the current analytical methods have less capability and less confidence to predict dynamic structural behaviors for structures with irregular than with regular configurations, especially with respect to torsion. The fact that buildings with irregular configurations have failed and collapsed more than buildings with regular configurations during earthquakes is a clear indication that irregular configurations pose problems for structural engineers, and building codes have issued warnings on the use of structures with irregular configurations.

- Irregular stiffness variations of the SC wall in both vertical and horizontal directions presents another challenge, which generates uncertain analytical results, especially with respect to torsion. Irregular stiffness variations is considered to be another one of the most undesirable design considerations, because not only the predicted stresses in the structure is unreliable but also the behavior of the structure is less predictable with a structure consists of irregular stiffness variations. Good design practices avoid irregular stiffness variations.

- The SC/RC connections are new types of connections: These connections have no precedents and have not been tested. Therefore, the stiffness variation of the connections along the zigzag boundary conditions with respect to the increasing levels of loads, such as SSE events, is unknown. WEC used [redacted] computer code to analyze the shielding building seismic behaviors without considering the existence of SC/RC connections due to the difficulty of modeling the stiffness of the connections. Therefore, the variation of stiffness of the SC/RC connections on the effect of dynamic behaviors of the shield building is unknown and cannot and has not been accounted for in the analysis and design.

- Based on the above discussions, the analysis results should be viewed as approximations at the best, and not as accurate representations of the actual behavior of the shield building during the SSE event. The highly irregular configurations, the stiffness variations in both vertical and horizontal directions of the shield building, and the SC/RC connections effect, have prevent precise prediction for the behavior of the shield building, especially with respect to torsion, when it is subjected to earthquakes. These imprecise predictions of stresses, strains, stiffness, and deformations in analysis must be recognized, considered, and taken care during the design stage. The inferiority and imprecise nature of using commercially available general purpose computer codes is address in Section 10 of this report.

Design:

- The behavior of the SC wall module subjected to impact force or energy, or seismic force or energy is unknown, because it is a new type of construction. WEC proposed a testing program to address some of the problems, [redacted] which are considered to be the major problems of the shield building design. However, tests for the SC wall module under the combination of [redacted]

- As discussed above, the highly irregular configurations, the stiffness variations in both vertical and horizontal directions of the shield building, and the SC/RC connections effect will produced torsion, and its actual magnitude is not only difficult to obtain but the behavior of the SC module reaction to torsion is unknown because there is no test data available on the SC module under torsion. Torsion has been identified as one of the major causes of building failures and collapse in the literatures based on past earthquakes experience and records. This is because our analytical capability in torsion and knowledge in the behavior of structural elements (module) or structures reaction to torsion are the least compared to other types of structural characteristics, such as flexure, axial, and bending.

Recognizing the major problems of the shield building, as listed above both in analysis and design, and the difficulty or lack of technical capability to solve them with high confidence, the NRC and WEC realized and identified that ductility of structural modules to be the most single important characteristic or material property that can compensate for these undesirable design considerations. Ductility also is needed to overcome the deficiency of analytical capability for the SC module behavior with respect to torsion, and the unknown effect of the SC/RC connections on the dynamic behavior of the shield building. As can be seen in Figure 1 below, ductility is a good representation of the energy absorption/dissipation capability of a structural module or structure, which is the single most important characteristic that can prevent a structure from being collapsed when it is subjected to impact from tornado missiles or aircraft, or ground motions from earthquakes. As can be seen in Section 6.1 below, building codes require ductility or energy absorption/dissipation capability to be designed or demonstrated through testing for structures subjected to earthquakes or impact from missiles. Since the building codes design methods were developed from tests of monolithic RC structural modules, and therefore are not explicitly applicable to the SC wall modules, [redacted] the testing program was focused on ductility.

3.0 Strength, Stiffness, Ductility, and Energy Absorption/dissipation of a Module or Structure

3.1 Test data of module #2 and RC module with nearly identical shear reinforcement [redacted]

Test data of module #2 and a RC module are presented in Figure 1 below to explain the definition of strength, stiffness, ductility, and energy absorption/dissipation capability of a structural module or a structure, and illustrate the different behaviors between the SC module #2 and the RC module.

[redacted]

Description: Red color is the load-deflection curve of a SC module with [redacted]
Blue color is the load-deflection curve of a RC module nearly identical to the SC module.

Figure 1: Load-Deformation Curves for Nearly Identical SC and RC Modules

Strength (How Strong is the Structure?): The highest point on the Load–Deflection curve, which is a measure of capability against applied load (force).

Stiffness (How much is the Deflection?): The slope from the origin 0 to any point on the Load-Deflection curve is a measure of capability against building deflection (a building must be stiff enough to meet building code drift limit. Stiffness is required to calculate relative building movements for piping design running through them). The stiffer the slope of the line, the less deflection the module or structure would be under the same load.

Ductility (How tough is the Structure?): Ductility is generally represented by displacement ductility ratio, and the displacement ductility ratio is calculated by the displacement of a module or structure at failure divided by the displacement of the module or structure when steel starts yielding. The higher the displacement ductility ratio, the more ductile the module or structure is.

Energy absorption/dissipation capability: The energy absorption/dissipation capability is measured by the area between the curve and the abscissa (blue vertical lines or red slant lines). The greater the area, the more energy absorption / dissipation capability the module or structure possesses. This is the most important characteristic or material property of a module or structure that indicates its capability to resist exterior energy imparted to it from ground motions of earthquakes or impact from missiles or aircrafts. As can be seen in Section 6.1 below, that the ACI Code's design requirements for earthquake-resistant structures and for structures subjected to impact loads from tornado missiles and aircraft missiles are based on the energy absorption/dissipation capability concept.

The above four characteristics are the measurement of superiority or inferiority of a structural module or structure. Structural modules possess high values of these characteristics are superior modules. As can be seen in Figure 1, the SC wall module #2 [redacted] The difference in this case could be due to that the [redacted] of module #2 was insufficient to tie the steel faceplates together and make the module acting as a unit. [redacted] This hypothesis was used as a basis to reject WEC's Revision 0 submittal where the module contained no [redacted] in the NRC October 2009 letter.

WEC subsequently tested SC module with [redacted] together and demonstrating the module acting as a unit, and thus attaining a displacement ductility ratio of [redacted]

3.2 The Conversion from Energy Absorption/Dissipation Capability to Ductility

Whether the shield building is subjected to the ground motions of earthquakes or impact from tornado-generated missiles or aircraft missiles, the shield building will not collapse if it possesses more internal energy absorption/dissipation capability than the external energy imparted to it. However, tests and calculations for energy absorption/dissipation capability are complex and difficult. Tests and calculations for ductility are simpler and easier. In addition, ductility is a good measure or representation of the energy absorption/dissipation capability of a structural module or structure, as can be seen in Figure 1, or a structure. Therefore the energy absorption/dissipation capability is usually represented or measured by tests for ductility.

3.3 Ductility Criteria Proposed by WEC for Out-Of-Plane Shear

Out-of-plane shear is the shearing force or energy that is acting perpendicular to the cylindrical wall surface of the shield building. Impact from tornado-generated missiles or aircraft missiles to the shield building wall creates out-of-plane shearing force and energy. Any restraint to the shield building wall, whether global or local, creates out-of-plant shearing force to the shield building wall due to thermal growth, or uneven thermal growth, within the structural module or structure, or earthquakes.

[redacted]

[redacted]

3.4 Out-of-plane shear test results of Module #1 and module #2

Module #1 with tie-wire spacing [redacted] possesses a ductility ratio of [redacted] Module #2 [redacted], as can be seen in Figure 1.

4.0 Westinghouse Design Basis

WEC design basis for the SC wall is the American Concrete Institute (ACI) 349 "Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary"

5.0 NRC Acceptance Criteria

NRC accepts the ACI 349 Code in the SRP and endorses both ACI 349 Code and ACI 318 "Building Code Requirements for Structural Concrete and Commentary" for reinforced concrete (RC) structures." in Regulatory Guide 1.142. ACI 349 Code has for frame members is related to flexural strength of the designed member rather than to factored shear forces indicated by SSE analysis.

The intent of this requirement is to assure that the ductile flexural failure mode in steel yielding would be developed prior to the occurrence of brittle shear failure modes. ACI 318-08 Code has the identical requirement. This requirement is about ductility because shear brittle failure modes would be avoided or suppressed if the shear strength is designed to be greater than flexural strength of a structural element (module) or a structure, and thus the structural module or structure would avoid shear brittle failure modes and behave in a ductile manner. The Code is very specific on the shear strength design. The shear strength is not designed based on the shear forces from the SSE analysis, but to be greater than the flexural strength of the module or structure. The intent of this requirement is to design ductility into the module or structure so that it will not fail in a brittle manner, such as shear failures. The ACI Code requirement for this ductility design for out-of-plane shear is for seismic. The requirement for the ductility design for out-of-plane shear for impact, such as tornado-generated missiles or aircraft missiles, is 20% more than that for seismic, as stated below.

- For impact design, ACI 349-90, Section C.3.6 states,

  For flexure to control the design, ...the load capacity of a structural element (*the author notes that a structural element is the same as a structural module, such as the SC wall module*) in shear shall be at least 20% greater than the load capacity in flexure,....

- ACI 318-08 Code, Chapter 19 – Shells and folded Plate Members, Section 19.4.5 states

  The area of shell tension reinforcement shall be limited so that the reinforcement will yield before either crushing of concrete in compression or shell buckling can take place. *(The author notes that shear compression failure is a type of failure that is caused by crushing of concrete and is induced by shear, which is a brittle failure.)*

From the above ACI Code excerpts and explanations, it is clear that the intent and requirements of the Code are that the design of a structure should avoid or suppress all brittle failure modes so that the structure is capable of behaving in a ductile manner with large energy absorption/dissipation capability to resist the energy imparts to it through ground motions by earthquakes or impact from tornado-generated missiles or aircraft missiles.

6.1.1.1 Tie-wire spacing based on ACI Code prescriptive requirements

- Required [redacted] based on the ACI Code prescriptive requirement for a RC element subjected to earthquakes: the staff calculated the required [redacted] when the out-of-plane shear strength equals to the flexural strength, [redacted] based on the ACI Code prescriptive requirement.

- Required tie-wire spacing based on the ACI 349-90 Code prescriptive requirement for a RC element for impact resistance: the staff calculated the required [redacted] when the out-of-plane shear strength is 20% greater than the flexural strength, [redacted]

6.1.1.2 The most recent prescriptive requirement [redacted]

- NEHRP (National Earthquake Hazards Reduction Program) Recommended Seismic Provisions for New Buildings and Other Structures (FEMA P-750), 2009 Edition, provides additions to the ACI 318, 2008 version, Code. One of the additions in Section 14.2.2.4, Intermediate Precast Structural Walls, is to add a section 21.4.5 to the ACI 318-08 Code stating "Wall piers... shall have transverse reinforcements...Spacing of transverse reinforcement shall not exceed 8 inches..." The commentary of section C14.2.2.4 states that "ductility behavior is to be ensured".

Based on the above discussion, it is clear that SC module #2 does not meet prescriptive requirements of the ACI Code and NEHRP Provision for out-of-plane shear reinforcement (tie-wire).

6.1.2 Design to meet Code Intent through Testing

ACI 318-08 Code, Section 1.1.8 states, "A reinforced concrete structural system not satisfying the requirements of this chapter shall be permitted if it is demonstrated by experimental evidence and analysis that the proposed system will have strength and toughness equal to or exceeding those provided by a comparable monolithic reinforced concrete structure satisfying this chapter."

Since the new SC wall modules are not monolithic RC modules, WEC proposed the testing route to meet the Code requirement, and the NRC agreed to this approach. [redacted]

6.1.3 Module #2 does not meet ACI Code requirement

Since module #2 does not meet the prescriptive requirement of the ACI Code, and failed to meet the testing demonstration by the Code requirement, continuing the use of module #2 for the SC wall does not comply with the ACI Code, which is endorsed by NRC as a basis for design of structures like the shield building.

7.0 Design change for Module #2 Recommended by the two NRC Consultants

The two NRC consultants recommended that (1) either using module #1 for the entire SC wall, or (2) using a [redacted] that meet the ductility criteria to replace module #2 (see Attachment 2).

8.0 The author's position

Based on (1) the staff's calculation that a tie-wire spacing [redacted] in accordance with the ACI Code's prescriptive requirement for seismic, (2) the staff's calculation that a [redacted] is required to prevent a brittle shear failure of a RC module in accordance with the ACI 349-90 Code's prescriptive requirement for impact resistance, (3) the most recent NEHRP, Recommended Seismic Provisions for New Buildings and Other Structures (FEMA P-750), 2009 Edition, that placed a [redacted] limit at 8 inches, (4) the demonstrated brittle failure data of module #2, and (5) the experts' testing experience and knowledge in seismic shear design and their recommendations, the author agrees with the expert's recommended design change as stated above.

9.0 WEC Justification for continuing use of Module #2

WEC submitted its "Shield Building Behavior and Design Philosophy", dated September 3, 2010, to the NRC. The submittal attempted to address that the SC wall module #2 with [redacted] is still adequate although the [redacted] for that module indicated a brittle failure.

[redacted]

The author finds that this new design philosophy was inconsistent with what the NRC and WEC agreed upon approach, including the test modules and their acceptance criteria , it is inappropriate for WEC to treat the [redacted] because the two structural systems are fundamentally different both in stress generation and in behaviors. These differences are explained below first and then followed by addressing the three WEC assumptions.

9.1 The Fundamental Difference in Behavior between a Frame and a Cylindrical shear Wall

[redacted]

[redacted]

[redacted]

[redacted]

Figure 2. In-Plane Shear distribution Along the Height of the Wall

Due to the three-dimensional cylindrical shape of the shield building, any point in the shield building is subjected to both in-plane and out-of-plane shears concurrently and simultaneously during earthquakes. This is because this shear wall is not a planar wall, but a three dimensional cylindrical wall. Although there are test data on behaviors of SC beams and SC columns under loads, there is no test data on behaviors of SC modules subjected to both in-plane and out-of-plane shears. Therefore, the behavior of SC module under both in-plane and out-of-plane shears is unknown.

9.2 [redacted]

[redacted]

Therefore, WEC's design philosophy remains a philosophy only, and is not acceptable for the design of RC frames.

WEC applied the capacity design philosophy or a [redacted] approach to the shield building wall is not acceptable because the shield building is a three-dimensional continuum cylindrical shell structure with no beams, no columns, and no joints.

[redacted]. Therefore, WEC claims that these areas do not need ductility. However, ACI Code requires the level of ductility in structural modules (elements), as well as structural systems, to be designed commensurate with the level of seismic risk or assigned seismic structure category. As discussed above, for structures with high seismic risk or high assigned seismic category, the ACI Code not only limit the yield strength of steel to [redacted] but also limit the actual yield strength of steel over the specified yield strength to [redacted] to ensure that the RC structural module can be designed analytically or verified by test to the desired ductility level. The shield building certainly belongs to the high seismic risk or high assigned seismic structure category, because it is required to withstand a design basis seismic event. Therefore, there is no justification to accept the SC wall module #2 [redacted]

9.4 WEC Assumption (2) of One Failure Mode is Insufficient

The WEC assumption (2) is insufficient because it assumed the failure mode of the shield building is a cantilever beam type planar failure. However, in the three-dimensional cylindrical shield building, several types of possible failure modes exist, such as diagonal tension failure in concrete, diagonal compression failure in concrete, sliding shear failure along construction joints, and out-of-plane shear failure due to impact loads or energy from tornado-generated missiles or aircraft missiles. In fact that the shield building is not a standing alone cylindrical shell structure. The shield building is connected to the roof of the auxiliary building and walls, as described in Section 2.1 above. Therefore, there will be additional other types of failure modes. [redacted].

In an effort to demonstrate the adequacy of SC wall modules [redacted]
The approach is also inconsistent with the ACI 349 Code design philosophy, which is to assure that brittle failure modes should be avoided or suppressed so that the structure will behave in a ductile manner by designing the shear strength to exceed the flexural strength, as stated in Section 6.1.1 above. The Code also recognizes the unpredictable nature of earthquakes and uncertainties in structural analysis, and therefore specifically states that the factored shear forces indicated by SSE analysis should not be considered as a design limit so that engineers would not wrongly assume or justify that SSE analysis results should be considered as the limit for its design. This is because that the shield building wall not only needs to be strong enough to resist static load (force) from the 6.7 million pounds of water plus the weight of the roof, but also has to be ductile

enough to posses sufficient energy absorption/dissipation capability to resist the energy imparted to it from tornado-generated missiles or aircraft missiles, or earthquakes. This required ductility ratio was determined and agreed upon between the NRC and WEC, after a lengthy discussion, [redacted] close to the blue load-deformation curve as can be seen in Figure 1. However, the ductility requirement is not mentioned in the WEC new design philosophy, which is not acceptable.

9.5 WEC Assumption (3) of Compliance with ACI Code is Incorrect

The author finds WEC assumption (3) incorrect because the [redacted] spacing may be appropriate for frames with simple configurations to meet the [redacted] detailing requirement, in accordance with the ACI 349 Code, but to extend the application for frames to walls is not appropriate, because that a frame behaves differently from a shear wall in resisting lateral loads, as discussed before. As indicated in Section 6.1.1.2, NEHRP (National Earthquake Hazards Reduction Program), Recommended Seismic Provisions for New Buildings and Other Structures (FEMA P-750), 2009 Edition requires a detailing requirement for wall module to be 8 inches. As indicated in Section 6.1.1, based on the ACI Code design requirements, the [redacted] for seismic, and [redacted] for impact, such as generated by tornado missiles. Therefore, WEC has wrongly applied the detailing requirement for [redacted] and does not address the design requirement for [redacted]. Therefore, if the detailing requirements and design requirements are properly addressed, the SC wall should have much closer [redacted] proposed by WEC.

9.6 Conclusion

[redacted]. The WEC new design philosophy of capacity design or a strong column-weak beam approach is not applicable to RC frames based on the ACI Code. [redacted]

10. Reasons for Non-Concurrence

(a) SER justifications for accepting module #2 are not valid with respect to structural engineering principles

As mentioned in Section 2.2 in this write-up, "Major problems of the SC wall", there are major problems associated with the analysis and design of this shield building, such as the highly irregular configurations, stiffness variations, and unknown behavior of the SC modules. These problems cannot be easily eliminated or resolved. The single most important treatment for the problems is to design and build ductility into these SC modules so that it could compensate for, or overcome, these problems. These problems had been discussed with WEC at length, and led to the agreement of using testing and acceptance criteria to verify the required ductility of structural modules. After SC wall [redacted] the proper engineering solution is to use the SC module #1, which meets the ductility acceptance criteria, for the entire SC wall or use another module that meets the acceptance criteria.

However, SEB1 SER accepts the continuing use of this module #2 with a justification, as stated in Section 5, Conclusion, of the SER,

> Based on (1) demonstration of conservative strength and adequate cyclic behavior for the SC module with [redacted] (2) confirmatory analysis which identified locations of potential SC steel plate yielding, and (3) the analogy with ACI-349, Articles 21.3 and 21.4, which require ductile detailing only where demands are high and plastic hinges are expected to form, the staff finds the applicant's use of [redacted] to be acceptable.

With respect to the SER justification (1) demonstration of conservative strength and adequate cyclic behavior for the SC module with [redacted] the author provides his evaluation below:

- The SC module #2 has about [redacted] than the RC module that has nearly identical shear reinforcement (tie-wire) ratio as that of module #2, as shown in Figure 1.

- Whether the actual strength of a structural module is conservative or not depends on the actual magnitude of stress that is acting on it and the actual strength of that structural module possesses. The author will demonstrate below that the SER assertion is not valid. Section 4.3.3 of the SER states,

> The results shown in Figures 4-3 through 4-6 of the letter show that the in-plane concrete shear stress using LS-DYNA and ANSYS remain below 600 psi for critical design locations analyzed. The applicant stated that these results demonstrate that the in-plane shear stress is below the allowable shear stress of 0.85x800 psi (680 psi) in ACI 349, Section 11.7.5.

The margin of the in-plane shear from the Code allowable is [redacted]. As explained in [redacted] above, due to the three-dimensional cylindrical shape of the shield building, any point in the shield building is subjected to both in-plane and out-of-plane shears concurrently and simultaneously during earthquakes, and the in-plane shear is nearly constant along the entire height of the wall. This in-plane shear must be added to the out-of-plane shear at every structural module in the shield building.

- Section 4.3.5 of the SER states,

> [redacted]

With this uncertainty of stress distribution, and about [redacted] alone prior to the addition of the out-of-plane shear, it is difficult to justify the words "conservative strength" as stated in the SER justification (1). If the fact that the unknown behavior of SC modules under both in-plane and out-of-plane shears is also considered, there is no basis for a conclusion that there is conservative strength in structural module #2 while its out-of-plane shear strength is [redacted] less than a companion RC module. For seismic design, the strength is not critical but ductility is. Therefore, even if the lateral strength is uncertain or less than the applied lateral loads (force), the structure will not fail as long as it meets ductility requirement. [redacted]

With respect to the SER claim of "adequate cyclic behavior", the claim is inappropriate because adequate cyclic behavior should be determined at the required displacement ratio, which is 3 cycles at [redacted] as stated in the acceptance criteria in Section 3.3 above, not at or below [redacted]. Cyclic behavior is to determine how tough (energy absorption/dissipation capability) a structural module is and observe the concrete degradation after steel yielding, and therefore must be conducted beyond [redacted]

WEC increased all types of stresses from analysis results by 5% to account for accidental torsion, while the correct way is to apply 5% of the plan dimension of the building perpendicular to the direction of ground motion as an eccentricity to the building to calculate torsional stress, as stated in SRP 3.7.2, Section 11. Therefore, WEC method for torsion is incorrect and unconservative. Without correct stress values, the SER cannot claim conservative strength demonstration because whether the strength is conservative or not depends on the magnitude of stress the module will be acted upon.

With respect to the SER justification (2) confirmatory analysis which identified locations of potential SC steel plate yielding, the author provides his evaluation below:

- If one assumes that the failure mode of the shield building were a type of a cantilever beam with no shear failures, [redacted] then the steel plate yielding would occur because the analysis result was forced to fit the assumption. As explained in Section 9.4 above, there are several failure modes that could occur to the shield building. The NRC consultant identified a failure mode in Attachment 2 that is a shear failure proceeding the steel plate yielding. Failure mode to the SC wall due to tornado-generated missiles or aircraft missiles is an out-of-plane shear failure, as stated in the ACI Code cited in Section 6.1 above. In order to avoid such a brittle shear failure mode due to the impact, the ACI Code requires, "the load capacity of a structural element in shear shall be at least 20% greater than the load capacity in flexure", as stated in Section 6.1.1 above. Unless such a design is performed, as it is done in Section 6.1.1above, and the required shear reinforcement [redacted] is installed, module #2 wall will fail prior to the flexural failure of steel plate yielding.

- All the analysis results for the shield building are obtained from commercially available general purpose computer codes [redacted] These results are approximations at the best and are not as accurate as results obtained from computer codes that were developed specifically for the type of construction so that the constitutive laws of the structural modules are accurately represented in the computer code. Constitutive laws are the relationships between the applied loads and behaviors of a structural module. [redacted]

[redacted] Therefore, WEC used the commercially available general purpose computer codes for confirmatory analysis. The NRC consultant, who is an expert in computer codes for RC structural analysis, states "the inference is that rarely are good results obtained for difficult RC problems [redacted]" in Attachment 4. Therefore, it is important to recognize the sources of inaccuracy or inadequacy of analysis results from what types of computer codes, and make allowances for the inaccuracy during the design stage.

[redacted]

With respect to the SER justification (3), the analogy with ACI-349, Articles 21.3 and 21.4, which require ductile detailing only where demands are high and plastic hinges are expected to form, the staff finds the applicant's use of [redacted] to be acceptable, the author provides his evaluation below:

- Section 4.3.5 of the SER states,

> The staff finds that ACI 349-01, Article 21.4.2.2 is intended for moment frame structures and is not directly applicable to cylindrical shell structures such as the AP1000 shield building. [redacted]
>
> The staff also finds that the calculation of member forces for the design basis seismic loads for the shield building did not involve load reductions that invoke the formation of plastic hinges for the dissipation of energy. In addition, the applicant's own design methodology for the shield building, based on ACI-349-01, requires that shear strength capacity must be provided everywhere including the assumed hinge locations, which is done for the shield building, while providing sufficient strength in the plastic hinge regions to meet the calculated shear demands is not a requirement for the "capacity design" approach. For the above reasons, the staff finds that the applicant's design methodology for the design of the shield building to resist seismic loads is not, in a strict sense, a "capacity design approach".

[redacted]

[redacted]

Therefore, the three justifications provided in SER for the acceptance of module #2 only address: strength, confirmatory analysis, and detailing, all are not valid with respect to structural engineering principles. More importantly, the SER does not address how the demonstration of the SC wall portion, which is made of module #2, possesses sufficient energy dissipation capability (ductility or toughness) as required by the ACI Code either by prescript design method or by testing, as stated in Section 6.1 above.

(b) Module #2 does not meet ACI Code intent and requirements

The ACI Building Code requires ductility to be designed into a structural module and a structure commensurate with the seismic risk or required seismic structure performance. The higher the seismic risk or required seismic structure performance, the more ductility would be required to design into the structural module and structure. The Code requires ductility to be designed into a structural module and structure for impact resistance more than that for seismic events. Since the main purpose of the shield building is shielding tornado missiles and aircraft missiles, and the building should not collapse as a result of the impact energy as well as the energy input from seismic waves during earthquakes. [redacted]

The WEC submittal on the tornado missiles, and the staff's acceptance, are based on the assumption that the SC wall module behaved identical to RC wall modules and as ductile as RC wall modules, as evidenced in Attachment 3. If the staff allows the brittle SC wall module to be used as part of the shield building wall, then the staff's evaluation on the adequacy of the wall for tornado missiles would have to be re-evaluated based on the actual brittle failure material property of the SC wall module #2.

If a tornado-generated missile or aircraft missile hits a ductile wall, such as the RC wall shown in Figure 1, and causes local failures, such as punching a hole through the wall, forces or energy in the shield building can be redistributed around the failed region to other regions due to ductility of the wall and the building is unlikely to collapse. However, if the same missile hits the brittle SC wall module #2, the impact energy could shatter the wall as it does to a glass cup. Glass has good strength, stiffness, but low ductility similar to the shape of module #2, as shown in Figure 1. Force and energy redistribution from the point of strike to other regions for brittle structural modules is unlikely, and the shield building integrity cannot be assured. Using brittle structural modules to resist impact loading and energy is not consistent with structural engineering principles while using ductile structural module is. A brittle SC wall does not possesses good energy absorption/dissipation capability to resist the energy imparts to it from earthquakes either.

The ACI Code is endorsed by the NRC, which regulates the design for all reinforced concrete structures, including nuclear power plant structures in the United States, prohibits brittle failure modes for structural modules and structures. ACI Code provides minimum requirements for the design and construction of RC structures. However, SEB1 SER accepts the brittle SC wall module to be used as part of the SC wall, and ignored the Code' prohibition for using brittle structural modules. SEB1 SER has not provided justifications as to why its acceptance standard, which is lower than that of the ACI Code, is adequate.

(c) Setting an acceptance standard lower than that for other shield buildings is inconsistent with GDC 1

10 CFR 50.55a (a)(1) and 10 CFR Part 50, Appendix A, GDC 1, "Quality Standards and Records", requires the shield building to be designed to quality standards commensurate with the importance of the safety functions to be performed. The AP1000 shield building, in addition to the shielding purpose, also performs a containment cooling function by using the 6.7 million pounds of water in the passive containment cooling water storage tank (PCCWST) on top of the shield building. [redacted] However, the SEB1 SER lowered its acceptance standard for AP1000 shield building than for that of other types of shield buildings, which are designed in accordance with the ACI Code requirements. This action is not consistent with the intent or requirements of 10 CFR 50.55a (a)(1) and 10 CFR Part 50, Appendix A, GDC 1.

(d) Summary

The SC wall proposed for use in the AP1000 shield building is a new type of construction, which has never been designed and constructed in the United States. No design codes are available for the SC wall. The highly irregular configurations, the stiffness variations in both vertical (meridinoal) and horizontal (circumferential) directions, and the zigzag SC/RC connection boundaries, represent unfavorable design conditions that good engineering practices would try to avoid. Under so many unfavorable design conditions, after numerous meetings, the staff and WEC concluded that design against collapse of the SC wall and the shield building should be the objective. In order to accomplish this objective, a structure must possesses large amount of internal energy dissipation capability (toughness or ductility) to resist the exterior energy imparts to it from tornado-generated missiles, or aircraft missiles, or earthquakes. In addition to the above stated unfavorable design conditions, [redacted]

Since the ACI Code method for designing ductility into a RC structure is not applicable to the SC wall, WEC chose to qualify its SC wall modules through testing for ductility. Test modules and acceptance criteria were agreed upon between the NRC and WEC. Module #1, [redacted] attained a displacement ductility ratio [redacted] and meets the staff's acceptance criteria [redacted]. Module #2, with [redacted] failed in a brittle manner, [redacted]. The two NRC consultants, who are the leading authorities in seismic design, recommended against the use of module #2. They recommended that either use module #1 for the entire SC wall or use [redacted] that meets the ductility acceptance criteria.

WEC submitted justifications for the continuing use of module #2, dated September 3, 2010. The author found WEC justifications inadequate. However, the SEB1 SER finds module #2 acceptable. The author reviewed the SEB1 SER and finds its basis for accepting module #2 was different from that of building codes and is not consistent with structural engineering principles. The structural engineering principles are (1) using ductile structural modules for impact resistant structures, and (2) designing the ductility level into the structure commensurate with the required seismic performance of the structure, but avoiding brittle failure modes.

The SER basis is that module #2 is used in areas [redacted] where stress is low, and therefore ductility is not needed. Such a basis may be correct for static loads, but is inconsistent with seismic and impact design concepts and building codes requirements. Neither WEC nor the SEB1 SER has addressed the following four questions, as a result of using the brittle module #2: (1) what is the effect of brittleness of module #2 on the capability of the SC wall, both local and global, in resisting tornado-generated missiles or aircraft missiles if the missile hits the area made from the brittle structural module #2?, (2) what is the effect of brittleness of module #2 on the capability of the SC wall, in resisting the energy imparts to it from ground motions of earthquakes if [redacted] of the SC wall is made of brittle structural module #2?, (3) What is the justification for the SER concept on ductility, which concludes that ductility is not needed where the stress is low, different from that of structural engineering principles and of ACI Code, which requires ductility for all structural modules in earthquake-resistant structures, as stated in Section 6.1 above?, and (4) how could the staff justify using a lower standard, by accepting a brittle structural module for about [redacted] of the SC wall for AP1000 shield building, which has more safety functions and greater consequence, if the wall collapses, than other types of RC shield buildings that are required to design to a higher standard of ACI Code?

Since there are inadequate technical justifications in the SER for accepting the [redacted] SC wall module #2 with respect to structural engineering principles, module #2 is not meeting the ACI Code ductility requirement, and the standard, which was used in the

Attachment 1

PRELIMINARY FEED BACK ON SB PART 1 REPORT
4/09/2010

SB PART 1

1. [redacted]

- [redacted]
- [redacted]

2. [redacted]

- [redacted]
- [redacted]

3. [redacted]

[redacted]

4. [redacted]

[redacted]

5. [redacted]

6. [redacted]

[redacted]

7. [redacted]

8. [redacted]

9. [redacted]

Attachment 2

DUCTILITY IN SHEAR RESPONSE OF SHIELD BUILDING

Frank Vecchio
July 9, 2010

Code Requirements:

The intent of Chapter 21 of ACI-349 (Provisions for Seismic Design) is that a structure be designed to behave, at its ultimate limit state, in a ductile manner. This necessitates that the structure exhibit a failure mechanism that involves yielding of the principal reinforcement such that a high deformation capacity be achieved, providing sufficient energy dissipation and avoiding all brittle failure mechanisms. The Code does not provide specific guidance on the levels of ductility that are required.

A generally accepted criterion for assessing adequate ductile behavior does not exist amongst jurisdictions or design code worldwide, although many are working toward criteria based on the concept of displacement or ductility demand. The criterion proposed in New Zealand, for example, states that if the structure can withstand four cycles at four times the yield displacement with no more than a 20% decay in force capacity, then it is adequately designed to resist high seismic loading. It is understood that this is a highly stringent criterion, particularly when isolating the behaviour in a single member or joint. Given the size and nature of the Shield Building (SB), three cycles at a displacement amplitude of three times the yield displacement with no more than a 20% reduction in strength would, in my opinion, be adequate evidence of good ductile behaviour.

Although ACI-349 does not define specific target levels for ductility, it is worth noting that it implicitly requires that the failure mechanism be ductile regardless of the magnitudes of the actual design loads. Thus, for shear design of flexural members, Clause 21.3.4.1 states that "...the design shear force shall be determined from consideration of the statical forces on the portion of the member between the faces of the joints. It shall be assumed that moments of opposite sign corresponding to the **probable flexural moment strength** act at the joint faces...". In other words, the design shear force is dictated by the flexural capacity of the member, ensuring that a ductile flexural failure occur before a brittle shear failure can develop. In the case of the SB wall, the ¾-inch steel faceplates create a large moment capacity, and thus the shear capacity required to maintain a ductile failure mechanism is high, regardless of the actual out-of-plane shear forces acting.

It is understood that the aforementioned requirements of Chapter 21 of ACI-349 relate primarily to moment-resisting frames, and not directly to shell structures such as the Shield Building. [redacted]

Nevertheless, Chapter 21 is based on the important underlying principle that if, in the event of unforeseen circumstances, the loads acting on the structure are of much larger magnitude than anticipated, then the structure should be able to achieve a ductile failure mechanism. [redacted]

[redacted]

[redacted]

[redacted]

To: Ma, John; Thomas, Brian; Patel, Pravin; Tegeler, Bret; Jain, Bhagwat
Subject: RE: The loose end

Hi, John:

After face-to-face discussion with you, I now fully understood your concerns, and recall the following review from my records:

The auto missile punch thru issue (auto wt. 4,000 lbs, impact speed 105 mph with impact area 6.6 ft x 4.3 ft) was analyzed in TR133, Rev.1 dated 5/28/2010 and APP-1000-CCC-015: "Nuclear Island—Tornado Missile Auto Impact above 30 feet"

The analysis considers [redacted]

Jerry

From: Ma, John
Sent: Saturday, September 04, 2010 11:51 AM
To: Thomas, Brian; Patel, Pravin; Tegeler, Bret; Jain, Bhagwat; Chuang, Tze-Jer
Subject: The loose end

Dear Colleague:

As the time of the SER for AP1000 is drawn near and the debate on the adequacy of the SC wall module with [redacted] has not been resolved and will likely be continuing among the staff, and that continuing debate creates a what I call a "loose end", which may spread to larger issues to tornado missile review and aircraft impact review. Since I have not been involved in the review of those areas and was assuming that the SC wall module with [redacted] would eventually be replaced by a ductile SC wall module, I need not and did not pursue the loose end. Now, it appears that the brittle failure SC wall may remain and therefore my original assumption would not be valid anymore. I am trying to present to you my view of the loose end, which is related to those review areas of tornado missiles and aircraft impact caused by the brittle failure of the SC wall module with [redacted] As you can see in the diagram below, the SC wall module subjected to an out-of-plane shear test, in red, has about [redacted] less in strength, [redacted] less in ductility, and [redacted] less in energy absorption/dissipation energy, than a nearly identical reinforced concrete (RC) module, in blue. The SC module will be used for about [redacted] of the SC wall. If a tornado missile or aircraft hits that [redacted] of the SC wall module, the missile or aircraft imparts a punching shear, which is a type of out-of-plane shear, and energy to the SC wall module, The loose end is that, during your review for the impact load (kinematic energy) form tornado missiles or aircrafts, has Westinghouse used the test data to calculate the punching strength and strain energy (the kinetic energy due to the deformation of the SC wall is near zero because the SC module has no ductility) to resist the punching shear force and Kinematic energy? If Westinghouse used computer codes for calculating the punching strength and strain energy, then those codes should include this particular test data of brittle failure of the SC wall module and not the ordinary reinforced concrete structure or steel structure modules (constitutive laws) into the computer codes.

The meaning and technical representation of ductility and a comparison of ductility between the WEC SC module with [redacted] and a nearly identical RC module are shown below:

[redacted]

Description: Red color is the load-deflection curve of a SC module with [redacted]

Blue color is the load-deflection curve of a RC module nearly identical to the SC module.

Stiffness (How much is the Deflection?): The slope measured from the origin 0 to any point on the Load-Deflection curve, which is a measure of **capability against building functionality** (To meet building code drift limit. To calculate relative building movements for the design of piping running through them).

Strength (How Strong is the Structure?): The highest point on the Load–Deflection curve, which is a measure of **capability against brutal force**.

Ductility (How Tough is the Structure?): A structure' toughness is measured by its **capability to absorb/dissipate seismic energy** imparted to it from earthquake ground motions or impact loads from missiles due to tornado or aircrafts. The **energy dissipation capability** is measured by the area under the curve between abscissa (blue vertical lines or red slant lines). **Ductility, toughness, and energy dissipation capability** are interchangeable.

[redacted]

Attachment 4

Ma, John

From: Tegeler, Bret
Sent: Saturday, September 25, 2010 10:19 PM
To: Ma, John
Cc: Williams, Joseph
Subject: FW: Reference Request - Correction

John,
Per your request. I believe this is the email that you are referring to. Let me know if I can do anything else.

Have a good trip to NY.
Bret

From: Pires, Jose
Sent: Monday, August 16, 2010 5:14 PM
To: Ma, John; Tegeler, Bret
Subject: RE: Reference Request - Correction

I just got some response from Frank Vecchio (please see below). Frank refers to SAP2000 below, which is not nearly as rigorous [redacted] for this type of nonlinear analyses. Frank recommends the following references:

1) Guner, S., and Vecchio, F.J.,2010, "Pushover Analysis of Shear-Critical Frames: Formulation," ACI Structural Journal, Vol.107, No.1, pp. 63-71, 2010.

The papers by Marti that he refers to are:

2) Thomas Jaeger and Peter Marti (2009), "Reinforced Concrete Slab Shear Prediction Competition: Experiments," ACI Structural Journal, Vol. 106, No. 3, pp. 300-308, 2009.

3) Thomas Jaeger and Peter Marti (2009), "Reinforced Concrete Slab Shear Prediction Competition: Entries and Discussion," ACI Structural Journal, Vol. 106, No. 3, pp. 309-318, 2009.

Given his response, I will be rephrasing somewhat what we had in our draft but not much (it is written below Frank's email).
==========================
Jose,

I hope you had an enjoyable vacation.

The publication policy of most journals is to not refer to, by name, any commercial product (in this case, software). Thus, you will not find many journal articles citing the deficiencies of commercial software. However, many of the recent state of the art reports (e.g., 2008 fib report) provide results that were obtained by software other than the large commercial packages; the inference is that rarely are good results obtained for difficult RC problems [redacted] etc.

One paper which specifically pointed the deficiency of SAP2000 is:

Guner, S., and Vecchio, F.J.,2010. Pushover Analysis of Shear-Critical Frames: Formulation, ACI Structural J., Vol.107, No.1, pp. 63-71

Another measure is from various blind Prediction Competitions held; e.g., the most recent one involved large-scale shear-critical slabs tested by Marti. Again, in such competition, rarely are the best results obtained with commercial packages.

So, there are no definite reports or studies addressing this issue. But the anecdotal evidence is compelling.

December 3, 2010

Enclosure 2

Redacted Version of the Staff Response to the Dissenting View on the Safety Evaluation Report on the Design of the AP1000 Shield Building

Staff Response to Dissenting View on the Safety Evaluation Report for the Design of the AP1000 Shield Building (SRP Section 3.8.4)[1]

October 17, 2010

Introduction

On September 27, 2010, Structural Engineering Branch (SEB) 1 of the U.S. Nuclear Regulatory Commission (NRC) issued its input to the final safety evaluation report (SER) for the Westinghouse Electric Corporation (WEC) design of the AP1000 shield building (Ref. 1). Dr. John Ma, the lead reviewer for the evaluation of the shield building design, decided not to concur in the SER and documented his position in "Dissenting View on the AP1000 Shield Building SER with Respect to the Acceptance of Brittle Structural Module to Be Used for the Cylindrical Shield Building Wall," dated September 27, 2010 (Ref. 2) (Dissenting View).

In accordance with the nonconcurrence process (draft Management Directive 10.158, "NRC Non-Concurrence Process"), the staff is providing (1) its summary of the dissenting view, (2) a summary of the staff's response to the dissenting view, and (3) a detailed response to the specific reasons provided in the Dissenting View, Section 10, "Reasons for the Non-Concurrence."

Summary of Dissenting View Regarding Staff's SER on AP1000 Shield Building

The fundamental reason for the nonconcurrence is disagreement with the staff's acceptance of WEC's use of a composite steel and concrete (SC) wall module [redacted] (referred to as module #2 by the lead reviewer). In tests performed by WEC, the SC wall module #2 failed in a brittle manner under out-of-plane shear with a ductility ratio [redacted] as shown in Figure 1 of the dissenting view.

Directly related to this aspect, the dissent identifies three major deficiencies in the staff's acceptance of module # 2: (1) inconsistency with the applicability of the American Concrete Institute (ACI) codes, (2) a safety standard in the SER that is less than that for existing or current proposed shield buildings, and (3) acceptance of an SC wall module that could lead to potential collapse. The following summarizes the dissenter's concerns:

(1) Inconsistency with Applicability of the ACI Codes

> The design is inconsistent with sound structural engineering principles and is inconsistent with the intent of the American Concrete Institute (ACI) Code, which the NRC endorses, that regulates the design for all reinforced concrete (RC) structures, including nuclear power plant structures. (Ref. 2, para. 1, p. 1)

[1] This response was prepared by B.E. Thomas (NRO/DE/SEB1), B. Tegeler (NRO/DE/SEB1), and J. Pires (RES/DE/SGSEB).

While the dissent raised all the above issues, primarily because of the dissenter's concerns for the uncertainties embodied in the WEC design and analysis methods, the dissenter acknowledges that he will concur in the SER if SC wall module #2 is replaced with a ductile SC wall module. This is reflected in the last paragraph of the dissent, as follows:

> Since there are inadequate technical justifications in the SER for accepting the brittle SC wall module #2 with respect to structural engineering principles, module #2 is not meeting the ACI Code ductility requirement, and the standard, which was used in the SEB1 SER for judging the acceptance of module #2, is inconsistent with the intent or requirements of 10 CFR 50.55a(a)(1) and 10 CFR Part 50, Appendix A, GDC 1, the author cannot concur in this SER. However, the author will concur in the SER, if an alternate design, replacing the brittle SC wall module #2, makes the entire shield building ductile. By ensuring the entire shield building wall to be ductile, the design meets the structural engineering principles, the intent and requirements of the ACI Code, and the intent and requirements of 10 CFR 50.55a(a)(1) and 10 CFR Part 50, Appendix A, GDC 1.

Summary of Staff Response to the Dissenting View

The AP1000 shield building is a first-of-a-kind nuclear power plant structure, for which there are presently no directly applicable design codes. However, the staff acknowledges that such structures can be designed and constructed to perform in a safe manner. For complex structures designed with or without applicable codes, engineers frequently use a combination of detailed analysis and testing methods that are based on sound engineering principles.

Sound engineering principles embodied in ACI 349, "Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary," issued 2001 (Ref. 8), aim to achieve (1) strength, (2) stiffness, and (3) ductility, consistent with a safe design for the specified design-basis demands. Furthermore, the premise of the code is that there is a balance among the above objectives. However, in the case of seismic loads, that balance is based on the assumptions made in calculating the demands on structural components. In the case of the shield building, the calculation of design forces does not assume ductility (toughness)-related energy dissipation, ACI 349 does not impose ductility requirements for these structures, and the demand/capacity ratios for out-of-plane shear for SC wall module #2 are small. Under these conditions, the staff found no justification in ACI 349 and ACI 318, "Building Code Requirements for Structural Concrete and Commentary," issued 2008 (Ref. 7), to require ductility requirements for module #2 for out-of-plane seismic shear forces. Two NRC consultants who have been leaders in the development of the earthquake engineering and RC design fields also could not find justification in ACI 349 and ACI 318 for ductility requirements for wall module #2 for out-of-plane seismic shear forces.

The dissenter focuses on the third goal as it relates to ductility detailing in module #2 for in-plane shear and, in fact, stated that he would concur on the SER if the staff revised the detailing [redacted]

WEC designed the AP1000 shield building using methods and tools consistent with guidance in NUREG-0800, "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants: LWR Edition" (SRP). The load cases and their various load combinations included the effects of dead, live, thermal, seismic, and aircraft impact. For seismic loading, the

AP1000 uses certified seismic design-response spectra that are consistent with NRC regulations and guidance (e.g., Regulatory Guide (RG) 1.60, "Design Response Spectra for Seismic Design of Nuclear Power Plants").

WEC used limit state equations consistent with ACI 349 and analyzed the demands consistent with the SRP. The analysis considered the effects of torsion and stiffness variation caused by the irregular geometry and SC/RC connections. Further, WEC conducted tests to confirm the applicability of the ACI limit state equations to the SC wall modules. These results showed that the criteria in the ACI limit state equations were met for the calculated demands. Confirmatory analysis verified the design and analysis assumptions. These analyses considered the effects of cracking and the resulting redistribution of forces on load demands and deformations. The staff viewed this as an acceptable approach.

Using the above approach, WEC demonstrated that, for design-basis loads (safe-shutdown earthquake (SSE)), the shield building structure remains essentially elastic and has margin beyond the SSE. These results show that, for the SC wall module [redacted] the maximum margins (design strength/demand) are as follows:

- in-plane shear [redacted]
- out-of-plane shear [redacted]

The staff notes that the dissent stated concerns with respect to maximum in-plane shear and maximum out-of-plane shear being additive and therefore reducing margin. However, the staff notes that the WEC analysis indicated that these maxima do not occur simultaneously at the same location and would not, therefore, be additive. The staff believes that there are additional sources of margin for the in-plane behavior of the SC wall module [redacted]

Based on the essentially elastic response of the shield building, small displacements, and demonstrated margin beyond the SSE, the staff finds that the design satisfies the regulatory requirements of Appendix S, "Earthquake Engineering Criteria for Nuclear Power Plants," to Title 10 of the *Code of Federal Regulations* (10 CFR) Part 50, "Domestic Licensing of Production and Utilization Facilities," which requires structures important to safety to be designed so that stresses, strains, and deformation remain within applicable deformation limits and are therefore safely designed to resist design-basis loads.

To assess the behavior of a shield building collapse, WEC performed a nonlinear pushover analysis. The pushover method is an accepted industry practice for estimating the limit state (i.e., collapse load) and corresponding mode(s) of failure of a structure caused by seismic loading. The response modes of the analysis models were not constrained and were capable of in-plane and out-of-plane failure. Based on a detailed review of the WEC pushover analysis model, the staff finds that it was validated by comparison with relevant experiments (e.g., Purdue tests) and is therefore acceptable for estimating the response of the shield building to severe seismic demands. The response modes of the analysis models were not constrained and were capable of in-plane and out-of-plane failure. The results of the analysis indicated that the stresses, strains, and deformations remain within acceptable limits for the SSE and beyond.

Based on the above, the staff finds that the WEC approach of using a combination of ACI code, requirements, testing, and confirmatory analysis constitutes sound engineering principles and satisfies NRC regulations. This finding is based on the small strains and displacements predicted for the shield building under seismic loads, as well as the results of a nonlinear pushover analysis, which demonstrated capacity greater than the SSE and that collapse is not imminent at the design-basis loading.

Staff Response to Major Dissenting Issues

(1) Inconsistency with Applicability of the ACI Code

The staff agrees that ACI 349 provides a sound technical basis for the design of RC structures and recognizes, in the SER, that ACI 349 is not explicitly applicable to SC structures. However, for the design of the shield building, the applicant demonstrated that the ACI provisions could be applied. WEC applied capacity formulas to the design of the SC wall modules and used those formulas to verify reinforcement (i.e., the steel plate, [redacted] and structural reinforcement for load transfer at the critical connections, such as the ring girder/air-inlet region and the SC/RC connections) of the design.

To validate the use of the ACI code, the applicant performed nonlinear analysis and conducted a testing program to verify the behavior and determine the stiffness, strength, and ductility of proposed SC wall modules under monotonic and cyclic loads. In addition, the applicant reviewed international test data on SC wall modules (see Ref. 3, Appendix A) to confirm the adequacy of the assumptions used in the integrated design process, such as the assumption that the SC wall modules would function as a unit.

The staff believes that the design of the shield building embraces the underlying engineering principles of the ACI code by using the code formulas for strength and ductility in the design. The code does not specify a ductility level, nor does it specify that ductility should be in every single structural component of the structure.

(2) Safety Standard in the SER that Is Less than that for Existing or Current Proposed Shield Buildings

The staff's standard for review of the shield building is the NRC's regulations pertaining to the design of nuclear power plant structures in General Design Criterion (GDC) 1, "Quality Standards and Records," and GDC 2, "Design Bases for Protection Against Natural Phenomena," in Appendix A, "General Design Criteria for Nuclear Power Plants," to 10 CFR Part 50. Appendix S to 10 CFR Part 50 requires that, if an SSE occurs, certain structures, systems, and components will remain functional and within applicable stress, strain, and deformation limits. WEC demonstrated that it met these regulatory requirements by providing adequate stiffness and demonstrating that stresses remained below the applicable limits, including the ACI limits, with margin.

Regardless of the presence and function of the AP1000 PCCWST located on top of the shield building, the standards of review are based on whether the design of the structure meets the above regulations. Consequently, the staff disagrees with the dissenting view, which states that the level of safety is reduced below that of existing and currently proposed shield buildings. The staff finds that WEC demonstrated that the shield building, as designed, has ductility where it is needed and sufficient strength in areas where the structure is brittle, and, in those brittle areas, the structure will not be subject to loads beyond the design-basis loads.

(3) SER Accepts SC Wall Module that Could Lead to Potential Collapse

The staff believes that the shield building is designed such that collapse is not imminent at SSE-level loads because there is sufficient margin, and that the pushover analysis results demonstrate that the stresses, strains, and deformations remain within acceptable limits for an SSE and beyond.

Although the design margin for in-plane shear is [redacted] the staff believes that there are additional sources of margin for in-plane shear strength of the SC wall module #2 beyond those already in the ACI stress limits. [redacted]

To assess the behavior of a shield building collapse, WEC performed a nonlinear pushover analysis. The response modes of the analysis models were not constrained and were capable of in-plane and out-of-plane failure. The pushover approach is an accepted industry practice for estimating the limit state (i.e., collapse load) and corresponding mode(s) of failure of a structure because of seismic loading. The staff finds that the pushover analysis model used by WEC was validated by comparison with relevant experiments (in-plane, and out-of plane shear) and is therefore acceptable for estimating the response of the shield building to severe seismic demands. The response modes of the analysis models were not constrained and were capable of in-plane and out-of-plane failure. The results of the analysis indicated that the stresses, strains, and deformations remain within acceptable limits for the SSE and up to the review-level earthquake.

Based on the small stresses, strains, and displacements predicted for the shield building under seismic loads, as well as on the results of a nonlinear pushover analysis that demonstrated capacity greater than the SSE, the staff found that collapse is not imminent at the design-basis loading.

October 17, 2010-Draft

Staff Response to Dissenting View

Section 10, "Reasons for the Non-Concurrence," of "Dissenting View on the AP1000 Shield Building SER with Respect to the Acceptance of Brittle Structural Module to Be Used for the Cylindrical Shield Building Wall," dated September 27, 2010 (Ref. 2) (Dissenting View), describes the dissenter's reasons for nonconcurrence with the staff's position described in the safety evaluation report (SER) (Ref. 1). The staff reviewed the reasons for the nonconcurrence and organized the issues into the following general categories:

(1) strength design (margin) (composite steel and concrete (SC) versus reinforced concrete (RC) construction)

(2) tornado and aircraft impact

(3) modes of failure and global collapse

(4) confirmatory analysis

(5) ductility and applicability of American Concrete Institute (ACI) 349, "Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary," issued 2001 (Ref. 8)

(6) torsion and SC/RC connection

Strength Design (Margin)

Dissenting View Comment(s):

> The results shown in Figures 4-3 through 4-6 of the letter show that the in-plane concrete shear stress [redacted] remain below [redacted] for critical design locations analyzed. The applicant stated that these results demonstrate that the in-plane shear stress is below the allowable shear stress of 0.85x800 psi (680 psi) in ACI 349, Section 11.7.5.
>
> The margin of the in-plane shear from the Code allowable is (680-600) divided by [redacted] As explained in Section 9.1 above [Ref. 2], due to the three-dimensional cylindrical shape of the shield building, any point in the shield building is subjected to both in-plane and out-of-plane shears concurrently and simultaneously during earthquakes, and the in-plane shear is nearly constant along the entire height of the wall. This in-plane shear must be added to the out-of-plane shear at every structural module in the shield building. (Ref. 2, p. 18)
>
> Section 4.3.5 of the SER states, the staff also finds that the shield building structure, a complex cylindrical shell, distributes loads in a manner that differs from two or three dimensional frames and can be more uncertain.
>
> With this uncertainty of stress distribution, and about [redacted] margin for in-plane shear alone prior to the addition of the out-of-plane shear, it is difficult to justify the words "conservative strength" as stated in the SER justification (1). If the fact that the unknown behavior of SC modules under both in-plane and out-of-plane shears is also considered, there is no basis for a conclusion that there is

conservative strength in structural module #2 while its out-of-plane shear strength is about [redacted] less than a companion RC module. (Ref. 2, p. 18)

Staff Response:

The staff notes that the calculation results referenced by the dissenter are for locations corresponding to the SC/RC connection (at the 100 foot (ft) elevation) and at the interface of the east wall and auxiliary building roof. The SC wall module used for these locations is the module with [redacted] The SC wall module [redacted] is used at a higher elevation in the shield building. The Westinghouse Electric Corporation (WEC) document, "Presentation and Actions for WEC Meeting with NRC June 9 through June 11," dated August 3, 2010 (Ref. 4), provides demand versus capacity plots for out-of plane shear. These results show that, for the SC wall module [redacted] the maximum margin (design strength/demand) is [redacted] (mentioned in SER Section 5.0). The staff agrees with the lead reviewer's recommendation that, because of the three-dimensional cylindrical shape of the shield building, any point in the shield building is subjected to both in-plane and out-of-plane shears concurrently and simultaneously during earthquakes. However, the staff believes that the cylindrical shape of the shield building prevents these maximum shear forces from acting at the same location. The maximum values for these respective shear forces will occur at locations that are approximately 90 degrees apart from one another and therefore should not be additive.

Regarding margin for the out-of-plane shear design of the [redacted] SC module, the staff notes that, in its August 3, 2010, submittal, [redacted] that demonstrate that, if the contribution of concrete, V_c, is completely neglected, there is margin against design forces. The maximum demand versus capacity ratio for only the V_s contribution is approximately [redacted] for seismic loads.

The staff believes that there are additional sources of margin for the in-plane behavior of the SC wall module with [redacted] The August 3 WEC document provides the results of an in-plane test conducted at Purdue University. The test specimen was representative of the AP1000 SC module with tie bars [redacted] The test results showed that the peak load for the module was [redacted] while the strength computed by ACI 349 was [redacted] While the in-plane shear test of the SC wall module indicated substantial strength margin to design loads, the module was not tested to capacity; therefore, the test did not demonstrate that the SC module would not fail in a brittle manner under cyclic loading. In a report referenced by the applicant, the staff found that a Japanese test of scaled models of SC structures (with geometry similar to the AP1000 shield building design) had demonstrated sufficient ductility for cyclic in-plane shear loading [14]. SER Section 4.3.5.2 describes the staff's evaluation of the Japanese test data.

Regarding the dissenter's comparison of SC and RC behavior (Ref. 2, Figure 1), the staff finds the comparison and resulting conclusions to be inappropriate. Differences between the test specimens and loads make the comparisons invalid. Specifically, the RC test specimen had an axial force that increases the shear strength of the test specimen, as shown in the study "Shear Strength Decay in Reinforced Concrete Columns Subjected to Large Deflection Reversals," issued August 1973 (Ref. 10), from which the test data were taken. The RC test specimen also had a higher shear reinforcement ratio, which further increases the shear strength of the RC test specimen. When these two factors are taken into account, the calculated strength would be approximately identical for the RC and SC. In addition, the flexural reinforcement ratio for the RC specimen was less that for the SC specimen. Because of the smaller flexural reinforcement

Section 6.1.1above, and the required shear reinforcement (tie-wires) with spacing of 8.1 inches is installed, module #2 wall will fail prior to the flexural failure of steel plate yielding. (Ref. 2, p. 19)

Staff Response:

In relation to impact loads from tornado missiles, ACI 349 does not require a ductile failure mode. Instead ACI 349 requires that specific conditions, such as the conditions in Article F.3.6 of ACI 349-2007, be met to invoke a ductile response mode that would dissipate the energy from tornado missiles.

Specifically, Article F.3.6 of ACI 349-2007 states: "For flexure to control the design, thus allowing the ductility ratios or rotational capacities given in F.3.3, F.3.4 and F.3.5 to be used, the load capacity of a structural member in shear shall be at least 20% greater than the load capacity in flexure, otherwise, the ductility ratios given in F.3.7 or F.3.9 shall be used."

However, flexure-controlled, ductile design is not the only type of design allowed by ACI 349 for protection against impulsive or impactive effects. Specifically, Article F.3.7 provides the permissible ductility ratios to use for beams, walls, and slabs where shear controls the design, which include permissible ductility ratios for shear resisted by concrete alone.

The staff also notes that, even for module #2, flexural, ductile modes may be invoked for certain impact and impulsive loads, depending on the shear span ratios that apply to the impact or impulsive loads considered, as well as on the axial compressive loads induced. Based on ACI 349, a condition that would invoke a flexural failure mode is for the shear span ratios involved to correspond to shear strengths greater than 20 percent of the load capacity in flexure. This would be supported by the applicant's out-of-plane testing for module #2, which showed ductile behavior for shear span ratios greater than 5.0 under monotonic loading.

[redacted]

Modes of Failure and Global Collapse

Dissenting View Comment(s):

[redacted] The shield building is connected to the roof of the auxiliary building and walls, as described in Section 2.1 above. Therefore, there will be additional other types of failure modes. WEC does not address these additional failure modes. (Ref. 2, p. 16)

A structure shall be designed to behave at its limit state (failure) in a ductile manner, and to prohibit all brittle failure molds. This is because a ductile structure possesses much more energy absorption/dissipation capability than a brittle companion structural module or structure, as evidenced in Figure 1. The objectives are to avoid structural collapse in very rare extreme ground shaking and to provide reasonable control of damage to loss of function in critical facilities. (Ref. 2, p. 9)

For seismic design, the strength is not critical but ductility is. Therefore, even if the lateral strength is uncertain or less than the applied lateral loads (force), the structure will not fail as long as it meets ductility requirement. However, the structure will fail if the lateral strength is less than the lateral applied loads and the structure is brittle without ductility. This is the problem of the shield building because structural module #2 is brittle and its strength is less than that of RC module. (Ref. 2, p. 19)

[redacted] As explained in Section 9.4 above, there are several failure modes that could occur to the shield building. The NRC consultant identified a failure mode in Attachment 2 that is a shear failure proceeding the steel plate yielding. Failure mode to the SC wall due to tornado-generated missiles or aircraft missiles is an out-of-plane shear failure, as stated in the ACI Code [redacted] In order to avoid such a brittle shear failure mode due to the impact, the ACI Code requires, "the load capacity of a structural element in shear shall be at least 20% greater than the load capacity in flexure," [redacted] Unless such a design is performed, [redacted] and the required shear reinforcement (tie-wires) with spacing of 8.1 inches is installed, module #2 wall will fail prior to the flexural failure of steel plate yielding. (Ref. 2, p. 19)

Staff Response:

[redacted]

[redacted]

In Attachment 2, the U.S. Nuclear Regulatory Commission (NRC) consultant, based on demand-to-capacity ratios provided by the applicant, also assumed a proportional increase of the loads above the design-basis loads, to conclude that a corresponding proportional increase of out-of-plane shears and moments would lead to shear failure before a flexural failure. This conclusion by the NRC consultant does not account for tensile forces in the steel faceplates caused by the overturning moments. These tensile forces, when combined with out-of-plane bending moments, can result in yielding initiation in the steel faceplates before out-of-plane shear failures, which is consistent with the results from the applicant's analysis.

The dissenting view expressed concern with respect to the global stability of the shield building. To address this issue, the applicant provided an analysis of global stability in its letter dated June 30, 2010. [redacted]

The staff reviewed the applicant's technical basis for global stability and found it to be consistent with the ACI Committee 334 report, "Concrete Shell Structures Practice and Commentary." The staff found the analysis to be acceptable based on an independent calculation of the critical buckling strength of elastic shells under compressive loads. As a result, the staff considers the issue of global stability and related Action Item 6 to be resolved.

Confirmatory Analysis

Dissenting View Comment:

> All the analysis results for the shield building are obtained from commercially available general purpose computer codes [redacted] These results are approximations at the best and are not as accurate as results obtained from computer codes that were developed specifically for the type of construction so that the constitutive laws of the structural modules are accurately represented in the computer code. Constitutive laws are the relationships

between the applied loads and behaviors of a structural module. [redacted] However, WEC has not performed sufficient tests to develop the necessary constitutive laws for the SC wall modules. Therefore, WEC used the commercially available general purpose computer codes for confirmatory analysis. The NRC consultant, who is an expert in computer codes for RC structural analysis, states "the inference is that rarely are good results obtained for difficult RC problems [redacted] in Attachment 4. Therefore, it is important to recognize the sources of inaccuracy or inadequacy of analysis results from what types of computer codes, and make allowances for the inaccuracy during the design stage.

Without realizing that WEC has not identified and analyzed all the possible failure modes, and that only one ductile failure mode was assumed using analysis tools not specifically developed for the SC wall, the SER's claim that the WEC confirmatory analysis had identified steel plate yielding as the failure mode is not valid. (Ref. 2, pp 19–20)

Staff Response:

The NRC staff and contractors use commercial finite element codes such as [redacted] to evaluate RC and steel nuclear power plant structures. In addition, U.S. Department of Energy laboratories, such as Brookhaven National Laboratory and Sandia National Laboratories, also routinely analyze nuclear power plant structures using these codes. WEC [redacted] models in a confirmatory analysis to predict the behavior of various elements of the SC module and compared those results to those established using the ACI 349 design methods and SC module tests. [redacted]

The applicant's design process for the shield building uses standard methods of analysis that meet the acceptance criteria in NUREG-0800, "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants: LWR Edition" (SRP), namely, linear elastic structural analysis, to calculate stress demands on the building. The confirmatory nonlinear analysis was used to confirm that concrete cracking and steel stresses would not cause significant changes in the design demands.

The staff notes that [redacted] are commercially available computer codes that are extensively used in the nuclear industry, as well as in other industries, to solve highly complex physical and numerical modeling problems, including the response of RC components under impact and impulsive loads. These codes have complex constitutive models, including a variety of models for concrete under multiaxial states of stresses, including cracking. [redacted] have reliable and tested nonlinear analyses algorithms and modeling features, such as contact between surfaces, that are useful for modeling components such as SC components. All three codes have been benchmarked for a variety of problems involving RC, including benchmarking [redacted] for impulsive loads conducted by the NRC staff, which provided a range of the capability of the codes to capture behaviors observed in tests. An NRC consultant, who is an expert on the modeling and analysis of RC structures, referred to reports that indicated relatively poor performance of some commercially available codes for certain benchmarking problems. Upon further clarification, this consultant referred to a comparison between the commercially available SAP 2000 and his own research code, as well as to state-of-the-art reports [redacted] from which the consultant inferred that the codes would rarely provide good results for difficult RC problems. The staff notes that SAP 2000 is not considered to be a code in the same category as [redacted] for modeling and analyzing RC structures and is used for different types of analyses and applications. The applicant did not use SAP 2000.

The staff recognizes that there are features in computer codes used for research applications and in codes specialized in the modeling of concrete structures that are not yet available in [redacted] However, the applicant benchmarked the models created using these commercial codes against results from its own testing program. The applicant noted that certain failure modes, namely, out-of-plane brittle shear failures, were not captured by its modeling approach (not necessarily a shortcoming of the codes themselves but of the modeling approach used in the benchmarking itself). In part for reasons such as these, the applicant defined limiting values for the tensile strains in the [redacted] to be tracked in the analysis so that the applicant would terminate the analysis when such strains first reached values at the onset of these brittle failures.

The staff evaluation of the applicant's benchmarking took into account possible sources of inaccuracies and defined the range of acceptability of the results of the analysis with the benchmarked models against their intended use. Based on this evaluation, the staff found that the models used by the applicant were adequate for load levels up to the safe-shutdown earthquake (SSE) to study redistribution of stresses from cracking and to assess strain limits and initiation of yielding, if any.

[redacted] the staff concluded that the applicant had [redacted] Further, the staff found that the applicant had adequately addressed the staff concerns raised in Action Items 12, 15, 16, and 17, as identified in the applicant's June 30, 2010, submittal.

Based on the above findings and the applicant's SSE load-level predictions of low stress and strain values in the SC steel plates, [redacted] the staff found the applicant's confirmatory analysis approach to be acceptable.

The analysis models also provide useful insight into the behavior and response of the structure as the loads are proportionally increased above their design-basis loads, a static pushover analysis, until the onset of significant yielding in structural components. This pushover analysis provides for a variety of external load distributions without constraining the modes of deformation and response of the structure. The various load distributions, assumed based on the design loads, account for a variety of loading directions, and each one is likely to be a conservative load distribution for the direction considered, at least up to the onset of yielding. Since the modes of deformation and response were not constrained, the analysis provides a useful indication of where yielding would start.

October 17, 2010-Draft

Ductility and Applicability of the ACI Code

Dissenting View Comment(s):

[redacted]

The ACI Building Code requires ductility to be designed into a structural module and a structure commensurate with the seismic risk or required seismic structure performance. The higher the seismic risk or required seismic structure performance, the more ductility would be required to design into the structural module and structure. The Code requires ductility to be designed into a structural module and structure for impact resistance more than that for seismic events. Since the main purpose of the shield building is shielding tornado missiles and aircraft missiles, and the building should not collapse as a result of the impact energy as well as the energy input from seismic waves during earthquakes. The required ductility ratio is 3.0, but module #2 possess a ductility [redacted] which is brittle and far from the required ductility acceptance criteria. (Ref. 2, p. 21)

The WEC submittal on the tornado missiles, and the staff's acceptance, are based on the assumption that the SC wall module behaved identical to RC wall modules and as ductile as RC wall modules, as evidenced in Attachment 3. If the staff allows the brittle SC wall module to be used as part of the shield building wall, then the staff's evaluation on the adequacy of the wall for tornado missiles would have to be re-evaluated based on the actual brittle failure material property of the SC wall module #2. (Ref. 2, pp. 21–22)

The ACI Code is endorsed by the NRC, which regulates the design for all reinforced concrete structures, including nuclear power plant structures in the United States, prohibits brittle failure modes for structural modules and structures. ACI Code provides minimum requirements for the design and construction of RC structures. However, SEB1 SER accepts the brittle SC wall module to be used as part of the SC wall, and ignored the Code's prohibition for using brittle structural modules. SEB1 SER has not provided justifications as to why its acceptance standard, which is lower than that of the ACI Code is adequate. (Ref. 2, p. 22)

With respect to the SER claim of "adequate cyclic behavior", the claim is inappropriate because adequate cyclic behavior should be determined at the required displacement ratio, which is [redacted] not at or below [redacted] Cyclic behavior is to determine how tough (energy absorption/dissipation capability) a structural module is and observe the concrete degradation after steel yielding, and therefore must be conducted beyond [redacted] (Ref. 2, p. 19)

Staff Response:

Ductility Requirements in ACI—Chapter 21 of ACI 318, "Building Code Requirements for Structural Concrete and Commentary," issued 2008 (Ref. 7), has ductility requirements for an earthquake-resistant design of certain types of RC structures and components, which are components of structures with a design basis that assumes and accepts excursions beyond yield. This approach is associated with reductions in the applied seismic loads that assume dissipation of the seismic energy imparted to the structure that relies on excursions beyond yield. For these conditions, ductility requirements and provisions are needed. These are the conditions that relate to the following comment in Section 6.1 (Ref. 2, p. 9) of the dissent taken from Chapter 21 of ACI 318: "The design and detailing requirements should be compatible with the level of energy dissipation (or toughness) assumed in the computation of the design earthquake forces." In the case of the shield building, the only energy dissipation mechanism assumed in the calculation of the design forces is 5-percent damping and no toughness (ductility)-related energy dissipation was assumed for the design-basis loads.

The design basis for the shield building does not invoke ductility-related energy dissipation to resist the earthquake forces. For its design-basis seismic loads, the shield building modules are designed not to yield under the design-basis loads and the seismic loads are calculated in a consistent manner. Therefore, the design basis for the shield building does not assume excursions beyond yield, and the provisions in Chapter 21 of ACI 318 are not applicable. ACI 349 also has ductility requirements that resemble those in ACI 318 for certain types of structures, namely for framed structures, although ACI 349 does not explicitly refer to components for which the design basis assumes excursions beyond yield. However, for structures like the shield building wall, Chapter 21 of ACI 349 does not specify ductility requirements for an earthquake-resistant design.

Even for those structures for which ACI 318 and ACI 349 require ductility provisions, these codes do not require that ductility be provided throughout the entire structure, for all components and all component failure modes, regardless of the demands imposed by the seismic loads. Examples are the ductility detailing requirements for framed structures (Articles 21.3 and 21.4). The applicant developed an analogy of the shield building design to this particular aspect of the code as part of its justification for not providing out-of-plane ductility requirements for module #2.

In the case of the shield building, the calculation of design forces does not assume ductility (toughness)-related energy dissipation, ACI 349 does not specify ductility requirements for these structures, and the demand capacity ratios for out-of-plane shear for module #2 are small and only reach a maximum value [redacted] in areas of the wall that are small and localized. Under these circumstances, the staff found no justification in ACI 349 to require ductility requirements for module #2 for out-of-plane seismic shear forces, nor did the two NRC consultants who have been leading the development of the earthquake engineering and reinforced concrete design.

The applicant chose to provide ductility detailing in those parts of the walls near supports and discontinuities (e.g., module #1), which are the regions with the most complex states of stress and where high out-of-plane demands are expected to develop under the earthquake loads.

The dissenter wrote that ACI 349 requires ductility to be designed into a structural module and structure for impact resistance even more than for seismic events. In fact, Appendix F to ACI 349-2007 only requires that ductility be designed into a structural module and structure for impact or impulsive loads if ductility needs to be invoked to resist these loads (Articles F.3.3 to F.3.6). If these conditions are not met, then ACI 349-2007 has provisions for a shear controlled, nonductile design to resist impact or impulsive loads (Article F.3.7). Those provisions are in Article F.3.7 and are consistent with the transient character and short duration of impact or impulsive loads.

Collapse—For the shield building, the design approach is that the components will not yield under all design-basis loads. WEC reported maximum design demand-to-capacity ratios [redacted] for in-plane and [redacted] (in localized areas of the wall) for out-of-plane shear. The staff notes, however, that the actual safety margin to loss of function, large deformations, or collapse at the design-basis SSE and as the loads increase beyond the SSE, will depend on the actual demand-to-capacity ratios and not simply on the limits in the ACI 349 prescriptive design equations.

The dissent states that the margin for in-plane shear is only about [redacted] Therefore, WEC provided evidence that the ACI 349 shear stress limits for in-plane shear are conservative for the SC wall modules, thus implying that there is in-plane shear capacity margin for the loads at the SSE level and as they increase beyond the SSE. This implies that the shield building wall will have reserve capacity at the SSE level and that loss of function or potential collapse are not imminent at the SSE level.

The nonlinear confirmatory pushover analysis conducted by WEC and addressed above (Ref. 2, p. 13) provides further indication that the stresses, strains, and deformations of the wall remain small and within acceptable limits and that loss of function or collapse of the wall are not imminent at the SSE level.

Acceptance Standards—The staff's standard for review of the shield building is the NRC regulations pertaining to the design of nuclear power plant structures (General Design Criterion (GDC) 1, "Quality Standards and Records," and GDC 2, "Design Bases for Protection Against Natural Phenomena," in Appendix A, "General Design Criteria for Nuclear Power Plants," to Title 10 of the *Code of Federal Regulations* (10 CFR) Part 50, "Domestic Licensing of Production and Utilization Facilities"). Appendix S, "Earthquake Engineering Criteria for Nuclear Power Plants," to 10 CFR Part 50 requires that, if the SSE occurs, certain structures, systems, and components will remain functional and within applicable stress, strain, and deformation limits. WEC demonstrated that these regulatory requirements were met by providing adequate stiffness and demonstrating that stresses remained below the applicable limits, including the ACI limits, with margin.

Regardless of the presence and function of the AP1000 PCCWST located on top of the shield building, the standards of review are based on the design of the structure meeting the above regulations. Consequently, staff disagrees with the dissenting view, which states that the level of safety is reduced below that of existing or currently proposed shield buildings.

Cyclic Loading—Regarding the dissenter's concern on cyclic loading, the staff finds, as stated on page 3.8.4-27 of the SER, that the applicant did not demonstrate that the SC wall module with [redacted] is ductile because the tests did not meet acceptance criteria for ductility as proposed by the applicant. However, the cyclic loading test for module #2 with a shear-span ratio demonstrated that the module can withstand various loading cycles at loads above the calculated design-basis loads, as well as at loads near or above the calculated design strength.

Torsion and SC/RC Connection

Dissident View Comment(s):

> As mentioned in Section 2.2 in this write-up, "Major problems of the SC wall", there are major problems associated with the analysis and design of this shield building, such as the highly irregular configurations, stiffness variations, and unknown behavior of the SC modules. These problems cannot be easily eliminated or resolved. The single most important treatment for the problems is to design and build ductility into these SC modules so that it could compensate for, or overcome, these problems. (Ref. 2, p. 17)
>
> Irregular configurations of the SC wall generate uncertain analytical results, especially with respect to torsion: As described above, the configuration of the shield building is highly irregular. Irregular configuration is considered to be the most undesirable design considerations, because the current analytical methods have less capability and less confidence to predict dynamic structural behaviors for structures with irregular than with regular configurations, especially with respect to torsion. (Ref. 2, p. 4)
>
> The SC/RC connections are new types of connections: These connections have no precedents and have not been tested. Therefore, the stiffness variation of the connections along the zigzag boundary conditions with respect to the increasing levels of loads, such as SSE events, is unknown. WEC used [redacted] computer code to analyze the shielding building seismic behaviors without considering the existence of SC/RC connections due to the difficulty of modeling the stiffness of the connections. Therefore, the variation of stiffness of the SC/RC connections on the effect of dynamic behaviors of the shield building is unknown and cannot and has not been accounted for in the analysis and design. (Ref. 2, p. 5)
>
> WEC increased all types of stresses from analysis results by [redacted] to account for accidental torsion, while the correct way is to apply [redacted] of the plan dimension of the building perpendicular to the direction of ground motion as an eccentricity to the building to calculate torsional stress, as stated in SRP 3.7.2, Section 11. Therefore, WEC method for torsion is incorrect and unconservative. Without correct stress values, the SER cannot claim conservative strength demonstration because whether the strength is conservative or not depends on the magnitude of stress the module will be acted upon. (Ref. 2, p. 19)

Staff Response:

The effects of stiffness variations and building asymmetries have been considered in the dynamic analysis and design of the AP1000 shield building. For design-basis seismic analysis, the AP1000 nuclear island (NI) was modeled with three-dimensional (3D) brick and shell elements using the [redacted] finite element code, with linear material properties, to obtain floor response spectra and input loads for structural analysis models. Confirmatory analyses using nonlinear material models for concrete (to account for cracking) and steel were performed using the finite element codes [redacted]. To address stiffness variations, the [redacted] model contained a simplified representation of the SC/RC connection, comprised of shell and brick elements. The properties of the elements in the SC/RC connection region were modified (i.e., benchmarked) to match the stiffness properties obtained from using the more detailed, Level 3 analysis models. A comparison of the linear [redacted] and nonlinear [redacted] analyses results showed that there were minor differences between the two codes and, therefore, demonstrated that the SC/RC connection has little effect on the dynamic response of the AP1000 NI. The WEC response to Request for Additional information (RAI)-SRP-3.8.3-SEB1-03 contains a more detailed description of the WEC approach. SER Section 3.8.3.2 describes the evaluation of RAI-SRP-3.8.3-SEB1-03.

The review of torsional effects is in accordance with guidance in SRP Section 3.7.2, Section 11. This SRP section states that an acceptable method to account for torsional effects in the seismic analysis of seismic Category I structures is to perform a dynamic analysis that incorporates the torsional degrees of freedom. WEC used 3D finite element models [redacted] to perform a dynamic seismic analysis. These models made use of shell and brick elements, which are capable of torsional (warping) degrees of freedom. The [redacted] models used the [redacted] which has shear deflection capability. SER Section 3.7.2 describes the staff's evaluation of these models.

Accidental torsion is intended to account for uncertainties in the calculated locations of the center of mass and the center of rigidity. SRP Section 3.7.2 states that, to account for accidental torsion, an eccentricity of +/- 5 percent of the maximum building dimension shall be assumed for both horizontal directions.

WEC, for seismic analyses, performed a response spectrum analysis using an input spectrum at the underside of the basemat that enveloped the response spectra for all soil cases, as well as for the hard rock case. The three directions of input were combined by the square root sum of the squares method and included a 1.05 factor on the horizontal components to account for accidental torsion. While the approach of applying a 1.05 factor to the horizontal seismic components is not fully consistent with SRP Section 3.7.2 guidance, the staff does not find the effects of accidental torsion to be significant for a building such as the shield building, which has a symmetrical cross-section geometry and mass distribution from the 179-ft elevation to the 327-ft elevation.

December 3, 2010

Enclosure 3

Redacted Version of the Rebuttal to Staff Response on Dissenting View on the Safety Evaluation Report on the Design of the AP1000 Shield Building the WEC push-over analysis for the shield building for the same reason as stated in Section 9.4 of the author's previous write-up, dated September 27, 2010.

The staff finding stated that the magnitude of loads and strengths in different areas of the shield building wall were properly demonstrated by WEC. However, evidence was not provided to substantiate that finding. NII has problems with the WEC stated strength, ductility, stress, analysis methods, and design methods, especially in shears in the SC wall and for tie-bars; as stated in the following questions. With respect to the problems related to strength, stress, and ductility, NII Question No.84 states, "The statement that the Shield Building is "...designed such that it exceeds the strength and ductility of RC structures" makes no sense.," and "WEC note that "SC plate structure shows great ductility" and "Air inlet area shows good ductility and lower stresses for all load cases. On what basis are the statements regarding ductility made? What are lower stresses?" Question 107 states "How is it possible to sustain such a high stress in concrete if nonlinear constitutive behaviour has been considered? There are also large areas shown with tensile stress in the order of 150-180 Ksf. Similar comments apply to Figures B-98 to B-103, B-140 to B-145 and B-188 to B-192". With respect to the design procedures, NII question Nos. 55 and 60 states, "What is the design procedure for out-of-plane shear where the loading effects include SSE shaking? How is interaction between in- and out-of-plane shear considered?" and "Consideration to combined in- and out-of-plane shear should be given in the development of the acceptance criteria. The average shear stresses at the onset of cracking should be reported." With respect to failure modes of the SC wall, NII question No. 62 states, "The results of the tests indicate that a brittle failure plane across the top of the shear studs is potentially the governing failure mode rather than a ductile failure of the studs themselves. However, there is no discussion in this document of potential failure modes associated with longitudinal failure planes in the concrete itself. Checks undertaken to BS5400 Part 5 Cl. 6.3.3.2 indicate that this mechanism controls the capacity due to an absence of rebar crossing the critical longitudinal shear plane across the top of the studs." NII question 100 states. "Checks should be performed for diagonal tension, diagonal compression and shear friction. What is the limit for shear friction? What is the limit for diagonal compression?" With respect to the WEC tie-bar design, NII question No. 8 states,

> "In the design of the enhanced shield building the [redacted] are spaced [redacted] and span transversely between the faces of the steel plates. We have concerns that various structural roles they perform are not properly reflected in the design methodology. We have identified four co-existent loading mechanisms that we believe will generate stresses in the [redacted] that in our opinion need to be addressed in any rational design methodology.
>
> Wet Concrete Pressure - during casting the [redacted] will experience tensile stresses due to the wet concrete pressures developed behind the steel plates. These tensile stresses will be locked [redacted] post-curing and should be accounted for in the final design.
>
> Out-of-Plane shear - the [redacted] will be mobilised in tension when resisting out-of-plane shear in a similar manner to shear stirrups in conventional reinforced concrete design
>
> Interface shear – longitudinal shear stresses generated at the interface between the steel plates and the concrete core will be resisted by the combined action of the shears [redacted]

Splitting forces – there is evidence that splitting forces are generated at the connections with RC and elsewhere that create bursting stresses in the concrete leading to splitting failure without the presence of transverse ties.

We require a detailed explanation justifying why it is appropriate for these load actions to be treated independently when determining the utilisation of the [redacted]

With respect to problems related to ductility, NII question No. 75 states," Explain why ductile limit states govern after initial yielding. What is the failure hierarchy in the ESB? Identify which ductile limit states enable the ESB to achieve deformations that are 3 to 4 times the SSE deformations. Demonstrate how system behaviour at RLE shaking is governed by ductile limit states." And NII question states, "Why is a ductility factor imposed if the failure mode can change from ductile to brittle through the use of excessive reinforcement? The second paragraph in this section is not clear. Why will out-of-plane shear not control the margin and what "shear stress exceeds yield"? A "margin factor" calculation should consider both normal forces acting on the plate, not just the circumferential force. NII question No. 102 states, " Why is a ductility factor imposed if the failure mode can change from ductile to brittle through the use of excessive reinforcement? The second paragraph in this section is not clear. Why will out-of-plane shear not control the margin and what "shear stress exceeds yield"? A "margin factor" calculation should consider both normal forces acting on the plate, not just the circumferential force."

With respect to problems related to computer codes, NII question No. 128 states,

"It is noted in Figure 8.7-7 and 8.7-15 that the test results exhibit significant and rapid unloading, but the FE analysis does not.

This discrepancy is poorly excused by the preamble in section 8.5, p 8-17 (see point 121).These tests seem to indicate a poor agreement between testing and analysis, but this is not discussed. In 8.7-7 a kinetic energy jump has been identified close to the test failure point, notwithstanding that the structure continues to have a positive gradient on the load-deflection plot.

When acting as part of a larger model, the important behavior will be the actual load deflection plot and not the existence of a small kinetic energy jump. We are concerned that the existence of this kinetic energy jump might be used to justify some sort of correlation between test and analysis. But if the curve does not follow the test then the correlation does not exist. Please see point 121. Several other problems related to WEC computer code analysis problems can be seen in the attached file."

BNL performed a detail review on the WEC analysis by LS-DANA computer code for the shield building, and concluded that "Based on the detailed evaluation conducted, there is no confidence that an appropriate level of quality assurance was implemented in the conduct of the LS-DYNA analyses". The report is attached.

The need and problem of reevaluation of the SC wall for tornado missiles and aircraft missiles

The reviewer on tornado missile for the AP1000 stated that his review for SRP Section 3.5, tornado missiles, is only applicable to concrete structures (See attached e-mail) and not to the SC wall. The lead reviewer for AIA indicated that a review for aircraft missiles taking into

Attachment 2

[redacted]

[redacted]

[redacted]

[redacted]

2. Demonstrate that the analysis model would also predicts the perforation for the test cases 1 and 2 shown in the SMiRT paper SMiRT18-J05-1, "INVESTIGATION ON IMPACT RESISTANCE OF STEEL PLATE REINFORCED CONCRETE BARRIERS AGAINST AIRCRAFT IMPACT PART 1: TEST PROGRAM AND RESULTS"

3. Demonstrate that WEC model also predicts perforation in RC structures for the known cases for which semi-empirical formula predict perforation.

Staff (Bret) and or Lupe (SNL) need to confirm if WEC provided and the staff reviewed in detail the above documentation. In absence of this confirmation, one can not reasonably conclude that the applicant used the validated computer model and the code for the aircraft impact class of problems to predict perforation including brittle failure to the SC structures. Results from such analyses are potentially suspected.

<u>Comment on realistic design specific aircraft impact assessment:</u>

The design margin in the shield building roof, tension ring and the PCCWST tank for SSE load is less than 2.0 (APP-1200-S3R-003, Rev. 2). The max acceleration at the shield building tank/roof level is of the order of 2g. The acceleration due to aircraft impact is of the several magnitudes higher than 2g. [redacted]. Assumptions of SSCs (water distribution bucket and other passive cooling components, e.g., air baffle, air diffuser) that are hung/supported-off the shield building roof remain intact following the impact is not a realistic assumption. In evaluating such scenario, SSCs within x ft distance in all directions of the impact should be considered immediately shock damaged and incapable of performing their intended function. This could potentially affect the air-only cooling and water cooling capability of the containment.

Attachment 3

Technical Evaluation

AP1000 Shield Building Design Report, Revision 2

[redacted]

(Prepared by R. Morante, M. Miranda, J. Nie, BNL, 05/30/2010)

The staff's technical assistance contractor, Brookhaven National Laboratory, evaluated the information in Rev. 2 of the SB report that describes the geometric and material modeling, the solution method, the numerical results, and the conclusions for the [redacted].

Finite element models are becoming more geometrically complex; material models (particularly for concrete) are becoming more sophisticated in an attempt to accurately simulate physical behavior; and highly specialized solution schemes are being employed to track the nonlinear response versus load. To ensure the validity of the results, the level of self scrutiny, independent checking, peer review, and benchmarking needs to be extremely high. The technical challenge is that the number of capable analysts, checkers, and peer reviewers decreases, as the complexity of the analysis increases.

Based on the detailed evaluation conducted, there is no confidence that an appropriate level of quality assurance was implemented in the conduct of the [redacted] analyses. [redacted].

[redacted]:

[redacted]

(3) Use of concrete material behavior models that may be inappropriate for static analyses intended to represent cyclic dynamic loading (i.e., earthquake); the effect of load cycling on the effective stress-strain relationship apparently is not considered. [redacted]
(4) Inadequate description of how the explicit dynamic solution approach, with mass scaling, has been implemented to simulate a static analysis, and how the progress of the solution is checked to ensure (a) no dynamic component to the response, and (b) no drift from the equilibrium path. [Q-AppB-4]
(5) Numerous confusing, misleading, or erroneous statements in the text of the SB report, related to the [redacted]

(6) Presentation of numerical results in tables, X-Y plots, and contour plots that are either obviously erroneous or highly questionable. [redacted]

[redacted]

In Section 10.2.1.5, on pages 10-27 and 10-28, the following statements are made:

[redacted]

Tension Ring and Air Inlet Stress Comparison

On page 5-25, Section 5.2.8, "Summary of Peak Stresses from LS-DYNA Nonlinear Analyses for SSE Level Loading", states the following: "To confirm the conservatism of the design approach described in this Chapter for the design loads including SSE, a series of nonlinear LS-DYNA finite element analyses were performed as described in Section 10. The analyses take into account the load re-distribution due to concrete cracking, concrete crushing, and steel yielding. Peak stresses at SSE level loading are illustrated in Figure 5.2-11 through Figure 5.2-15. The results are summarized in Table 5.2-6."

Table 5.2-6, "Comparison of Level 2 and LS-DYNA Analyses Results for SSE Level Stresses", presents a comparison of stresses between "Standard Level 2 Analyses" and "Confirmatory LSDYNA Analysis" for both the tension ring and air inlets. The stress comparisons in this table indicate that the LS-DYNA results are significantly lower than the standard level 2 results for the air inlets, by a factor of 3 for stress in the ties, and over 5 for "circumferential" and "vertical" stress. [redacted]

Westinghouse needs to explain why there is such a drastic reduction in the predicted stresses. Demonstrate that at low load, prior to the onset of concrete cracking and crushing, and steel yielding, there is reasonable correlation between the 2 solutions. Describe the redistribution as the load is increased, from the linear state at low load to the apparently highly non-linear state at peak load?

[redacted]

[redacted]

Table 2.6-1 Levels of Analysis

On page 2-17, Table 2.6-1 has 2 columns labeled "Standard" and "Confirmatory", and 3 rows labeled Level 1, Level 2, and Level 3. The staff notes that there are several errors and oversimplifications in this table. One example is identifying [redacted] as a nonlinear analysis. A second example is identifying that yielding is considered in a [redacted] nonlinear elastic analysis. A significant oversimplification in the table is that NI05 RSA is used for Standard Level 2 analysis. Correct and expand the table to accurately reflect what was done and the type of analysis conducted with each computer code/model.

[redacted]

Application of 100-40-40 Rule in [redacted] analysis

On page 10-29 of the Rev. 2 report, the 100-40-40 rule is applied to the seismic load input for a [redacted] equivalent static analysis that considers concrete cracking. The 100-40-40 rule is intended to be applied at the response level, not the load input level. In addition, only 4 of the possible 24 combinations are analyzed, without sufficient quantitative justification why these 4 cases are controlling. Justify use of the 100-40-40 rule at the load input stage, and also provide a clear and complete technical basis why only 4 cases need to be analyzed.

[Questions on Appendix B - [redacted]]

Q-App B-1:

[redacted]

(b) All numerical models of inelastic materials have inherent limitations. The analyst must exercise a significant level of judgment in order to capture the relevant physical behavior to be simulated, while at the same time, minimizing spurious numerical artifacts. It is not clear if a sufficient level of benchmark analysis has been performed to ensure that the material models used to simulate th[redacted] system in the Shield Building are appropriate for seismic loading conditions, i.e., inelastic behavior under cyclic loads with possible strength and stiffness degradation. Since a quasi-static pushover-type analysis is performed to simulate seismic response, the load-deformation characteristics of the concrete-steel plate-steel tie system should reflect the "backbone curve" typically observed under cyclic loading tests, and not necessarily the static load-deformation response under monotonic loads. Provide the results of benchmark analyses performed to demonstrate that the chosen material models and associated numerical parameters adequately represent the backbone curves of cyclic loading tests.

[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

Appendix B, pages B-9 through B-114 present tables and figures of results for load cases 1 through 4 without any discussion of significance or conclusions.

Tables B-3 and B-4 summarize specific results for all 4 load cases. The following information in these tables requires further explanation:

[redacted]

The detailed results for each load case can be located using the information in the following table:

[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
[redacted]	[redacted]	[redacted]	[redacted]	[redacted]
[redacted]	[redacted]	[redacted]	[redacted]	[redacted]

The following information in these figures requires further explanation:

[redacted]

[redacted]

n. The Appendix B figures for Load Cases 2, 3, and 4 contain a number of similar errors and require further explanation of the same issues as for Load Case 1. As applicable, update the figures for Load Cases 2, 3, and 4, consistent with the staff's questions on Load Case 1.